FPMA

FloodPlain Management Assessment

June 1995

Appendix A
(Hydraulic Modeling)

US Army Corps
of Engineers

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The oversight agency for the floodplain management assessment was the North Central Division. The St. Paul District was the lead agency for completion of the report, but actual work on the report was accomplished by five Corps District; St. Paul, Rock Island, St. Louis, Kansas and Omaha.

The assessment evaluated the impacts of a wide array of floodplain policies, programs and flood damage reduction measures to the Midwest Flood of 1993. However, this assessment has taken an important step toward achieving a better understanding of the current uses of floodplain, forces causing those uses and impacts of various alternative changes in the managment of floodplains. Some of the objectives included in the assessment are: describing land and water resources and making projections of future conditions; identify local interests; alternative uses of floodplain resources; identify facilities needing additional flood proction; examine Federal cost-sharing; evaluate cost effectiveness of alternative flood control projects and recommend improvements to current system.

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FPMA
Floodplain Management Assessment
June 1995
Final Report
Hydraulics and Hydrology Appendix

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This appendix is organized by Corps of Engineers District. The UNET Modeling was performed system-wide, and modeling assumptions were consistent between districts. A main output was maximum stage reductions for the various alternatives. Each district's section has tables showing those stage reductions at gage locations along the reach that was modeled within their district. To arrive at those peak stage reductions, the entire 1993 flood hydrograph had to be modeled. The 1993 flood was modeled well within the accuracy that was required for this assessment.

FPMA

FloodPlain Management Assessment

Hydrologic Evaluation

Omaha District

June 1995

US Army Corps
of Engineers
Omaha District

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FPMA
U.S. Army Corps of Engineers Omaha District

Hydrology and Hydraulics
June 1995

1. Study Purpose.

The 1993 flood event resulted in catastrophic damages in large portions of the upper Mississippi River, lower Missouri River, and major tributaries. While existing forms of flood protection reduced or prevented damages to many properties, these measures often proved inadequate to withstand the magnitude of flooding experienced during 1993. Assessment objectives were established to correspond with specific directives provided in the Conference Report for the FY 1994 Energy and Water Development Appropriations Act, (House Report 2445), the House Resolution (Docket 2423, dated 3 November 1993) and the guidance memorandum prepared by the Headquarters, U.S. Army Corps of Engineers, dated 14 December 1993. The congressional docket instructed the Secretary of the Army to conduct comprehensive, system-wide studies to evaluate the flood control and flood plain management needs of the upper Mississippi and lower Missouri River and their tributaries that were flooded in 1993. The purpose of the Floodplain Management Assessment (FPMA) was further defined to assess the adequacy of current flood control measures on the upper Mississippi River and its tributaries. Within the hydrologic perspective, the study was intended to focus on identifying facilities which require additional flood protection, assess the adequacy of current flood control measures, evaluate the cost-effectiveness of alternative flood control projects, and recommend improvements to the current flood control system.

2. Study Scope.

A scope was developed for performing the requirements of the FPMA. Numerous objectives were established which address issues in areas including hydrologic, economic, environmental, floodplain management, and others. In response to hydrologic requirements of the FPMA, development of a comprehensive system wide modeling tool of the Missouri River, Mississippi River, and significant tributaries was required. An unsteady flow modeling tool was necessary in order to adequately assess impacts of floodplain management and assessment alternatives on a system wide basis. The unsteady flow model was constructed of the Mississippi, Missouri, and significant tributary rivers. Corps District offices along the Mississippi River include St. Paul, Rock Island, and St. Louis. Corps District offices along the Missouri River include Omaha and Kansas City. While coordinating with all involved Corps Districts, each unsteady flow model was developed independently. Assimilation of model results and system wide routing was then performed for all conditions examined between adjacent Districts. This report pertains to the hydrologic modeling and results performed by the Omaha District for the FPMA.

3. Study Area.

For purposes of the FPMA, hydrologic modeling was performed by the Omaha District along the Missouri River from Omaha, NE, at river mile 615.9, downstream to Rulo, NE, at river mile 498.0. Rulo, NE, corresponds with the Omaha District boundary with Kansas City District. An upper river model was also constructed from Omaha, NE upstream to Gavins Point Dam, at river mile 811.1, in order to assess

main stem reservoir alternatives. In 1993, Missouri River peak discharges upstream of Omaha, NE corresponded to less than a 10 year recurrence interval. The upper river model was constructed with less detail as the flooding was minimal. A general location map illustrating significant project features within the study reach is shown in Plate OM-1.

3.1. Basin Description.

The Missouri River originates in the northern Rocky Mountains along the continental divide and flows south and east to join the Mississippi River near St. Louis, Missouri. At 2,315 miles (1960 mileage), it is the longest river in the United States. The Omaha District encompasses approximately 414,900 square miles of the drainage basin upstream of Rulo, NE to the river headwaters in the Rocky Mountains. The Missouri River basin contains numerous reservoirs and impoundments constructed by different interest for flood control, irrigation, power production, recreation, and water supply. The most significant flood control project constructed within the basin are the six main stem Missouri River Dams constructed by the Corps. The six dams, which were completed by 1964, provide flood protection by controlling runoff from the upper most 279,000 square miles of the drainage basin. The reservoir system has a total combined capacity in excess of 73 million acre-feet of which more than 16 million acre-feet is for flood control. Gavins Point Dam, located near Yankton, SD, at river mile 811.1, forms Lewis and Clark Lake and is the most downstream of the projects.

3.2. Missouri River.

Between Omaha, NE and Rulo, NE, the Missouri River drainage area increases from 322,800 to 414,900 square miles. Average channel gradient within the reach varies from 0.8 to 1.2 feet/mile. Total valley width usually averages 5-10 miles between the bluffs. The Missouri River generally follows the right (west) bluff line. The Platte River, which enters the Missouri River downstream of Omaha, NE at river mile 594.8, is a major contributor of coarse grained sediment.

3.3. Navigation Structures.

There were seven acts of Congress which provided for the construction, operation and maintenance of a navigation channel and bank stabilization works on the Missouri River. The most recent was authorized in 1945 which provided for bank stabilization and a 9-foot deep and not less than 300 feet wide navigation channel for the Missouri River from its confluence with the Mississippi River at St Louis, MO to Sioux City, IA for a total distance of 734.2 river miles. This was accomplished through revetment of banks, construction of permeable dikes, cutoff of oxbows, closing minor channels, removal of snags and dredging.

In order to achieve the project objectives of bank stabilization and navigation, the river was shaped into a series of smoothly curved bends of the proper radii and channel width. Stabilization of the bank along the concave alignment of the design curve was accomplished with pile and stone fill revetments. Dikes were constructed along the convex bank, approximately perpendicular to the flow. These dikes were designed to prevent bank erosion and to promote accretion, forcing the channel to develop and maintain itself along the design alignment. In areas where the natural river channel did not conform to the design alignment, canals were excavated and natural channels blocked in order to force the river to flow along the design alignment.

Since the initial stabilization efforts under the jurisdiction of the Missouri River Commission in

1884 through completion of the navigation channel in 1981, the overall length of the river channel has been reduced. This is evident at Sioux City, IA when the 1890 mileage of 807.5 river miles is compared to the 1941 mileage of 760.0 river miles and the 1960 mileage of 732.3 river miles.

3.4. Tributaries.

The major tributaries to the Missouri River from Gavins Point Dam to Rulo, NE are described in the following paragraphs:

3.4.1. Big Sioux River - RM 734.23. The Big Sioux River is a major left bank tributary of the Missouri River which enters the Missouri River near Sioux City, IA at river mile 734.23. The basin has a drainage area of approximately 9,006 square miles and includes portions of South Dakota, Minnesota and Iowa. During 1993, the Big Sioux River at Akron, IA, downstream of the confluence of both Split Rock Creek and the Rock River, experienced a peak discharge of 66,700 cfs on May 10. This was the second highest discharge of record (1929-1993) for the Big Sioux River at Akron, IA.

3.4.2. Little Sioux River - RM 669.17. The Little Sioux River is a left bank tributary to the Missouri River that drains approximately 4,500 square miles above its confluence with the Missouri River at river mile 669.17. An extensive system of federal levees has been constructed in the lower basin on both the Little Sioux and its tributaries to protect primarily agricultural lands. The levee construction began in 1956 and was completed in 1966.

3.4.3. Soldier River - RM 663.97. The Soldier River is a small left bank tributary of the Missouri River located immediately to the north of the Boyer River Basin. It drains approximately 445 square miles of western Iowa and enters the Missouri River at river mile 663.97. A new peak discharge of record (1940-1993) of 23,400 cfs was reported for the Soldier River at Pisgah, IA (DA = 407 mi^2) on July 9. A preliminary statistical analysis of the Pisgah, IA gage using the 1993 peak discharge indicates that the 1993 flood had an estimated recurrence interval of 10-25 years.

3.4.4. Boyer River - RM 635.2. The Boyer River is a small left bank tributary to the Missouri River at river mile 635.21. It drains approximately 1,190 square miles of western IA. The most severe flooding within the Boyer River Basin occurred from intense rainfall on July 8-9, which produced a peak discharge of 26,300 cfs for the Boyer River at Logan, IA (DA = 871 mi^2) on July 10. A preliminary statistical analysis of the Logan, IA gage using the 1993 peak discharge indicates that the 1993 flood had an estimated recurrence interval of 10-25 years.

3.4.5. Platte River - RM 594.30. The Platte River is a major right bank tributary of the Missouri river draining an area of approximately 90,000 square miles of northeast Colorado, southeast Wyoming and most of central Nebraska. The Platte River joins the Missouri River approximately 21 miles downstream of Omaha, NE at river mile 594.30. In eastern Nebraska, major tributaries to the Platte River are Salt Creek, the Elkhorn and Loup Rivers. The lower Platte River and tributaries experienced significant flooding in 1993. Flooding in July on the Platte River was caused by excessive rainfall over many of the lower basin tributaries. Rainfall amounts totaling more than 14 inches fell in a three day period over parts of the basin with wide spread averages of 6 to 10 inches. An estimated discharge of 160,000 cfs on the Platte River at Louisville, NE on July 25 established a new record and contributed greatly to the record flooding on the Missouri River downstream of Nebraska City, NE. Flooding along the lower Platte River was primarily confined to the lower 28 river miles of the basin downstream of Ashland, NE. Using the 1993 event and the historic record, a preliminary statistical analysis indicates that

the July event has a recurrence interval of approximately 25-50 years.

3.4.6. Weeping Water Creek - RM 568.7. Weeping Water Creek is a right bank tributary to the Missouri River located in southeast Nebraska at river mile 568.70. It has a drainage area of approximately 240 square miles above the USGS gaging station at Union, NE. Weeping Water Creek established a new record stage at Union, NE of 30.97 feet with a discharge of 65,100 cfs on July 23. Using the 1993 event and the historic record, a preliminary statistical analysis indicates that the July event has a recurrence interval of approximately 50-100 years.

3.4.7. Nishnabotna River - RM 542.08. The Nishnabotna River is a major left bank tributary to the Missouri River located approximately 20 miles downstream of Nebraska City, NE at river mile 542.08. It has a total drainage area of 2,995 square miles. Major changes within the basin include the construction of federal levees, private agricultural levees, channel changes and drainage improvements. The Nishnabotna River has federal levees along the right bank from the Missouri River confluence to Highway 275 upstream of Hamburg, IA. The left bank has federal levees from the Missouri River confluence upstream to Highway 275 south of Hamburg. Damaging floods have been reported within the Nishnabotna River Basin as far back as 1849. From 1883 through 1958 there were at least 24 lives lost to flooding. As the Nishnabotna River stages rose to record levels on July 25, a flood fight effort prevented overtopping of the Nishnabotna tie-off to federal levee L-575. Although the estimated peak discharge of 38,000 cfs at Hamburg was the second highest of record, the peak stage exceeded the previous high by over 4 feet due to backwater effects from the Missouri River, sedimentation on the berms, and private levees constructed within the floodway.

3.4.8. Little Nemaha - RM 527.8. The Little Nemaha River located in southeast Nebraska is a right bank tributary of the Missouri River. The basin extends over five counties, has a drainage area of approximately 885 square miles and enters the Missouri River at river mile 527.80 near Nemaha, NE. The peak stage of 26.49 feet with a discharge of 105,000 cfs on July 25 is the second largest of record. Frequency estimates based on a preliminary analysis of gage statistics, including the 1993 event, place the recurrence interval of the July 1993 event at approximately 25-50 years. Flooding of agricultural lands was extensive. The Missouri River federal tie back levee R-548, located along the Little Nemaha River at Nemaha, NE, was overtopped. Although the levee was overtopped and experienced some damage, it did not fail.

3.5. Mainstem Missouri River Levee System.

The Missouri River levee system was authorized by the Flood Control Acts of 1941 and 1944 to provide protection to agricultural lands and communities along the Missouri River from Sioux City, IA to the mouth at St. Louis, MO. The levees were designed to operate in accord with the six main stem dams. The extent of the levee system within the Omaha District consists of levee units on both banks from near Omaha, NE to Rulo, NE. Although many federal levees were proposed north of Omaha, NE along the Missouri River, few have been built due to the significant contribution of the main stem dams in this reach and the reduced impact of the Platte River. The majority of the area planned for protection by federal levees, north of Omaha, NE, is protected by private or non-federal levees with varying degrees of protection.

3.5.1. Federal Levees. A system of federal levees exists from Omaha, NE, to near Rulo, NE. Levees were constructed as part of local flood protection projects in the larger metropolitan areas of Omaha, NE and Council Bluffs, IA. The remainder of the federal levees were constructed as part of

the Missouri River basin Comprehensive Plan to protect smaller communities and agricultural lands. All of the levee units on the Missouri River were designed to operate in conjunction with the six main stem dams to reduce flood damages as part of the Pick-Sloan plan. For the purposes of the FPMA, modeling efforts were confined to the reach from Gavins Point Dam to Rulo, NE, within the Omaha District.

Federal levees were constructed in the 1950's and are usually set-back from the river bank a distance of 500-1500 feet. Survey data for the federal levees consists of 35 to 45 year old profiles or as-builts. Federal levees cover the left bank from river mile 515.2 to river mile 619.7. Levees on the right bank are intermittent since the river is often near the bluff. Total federal levee length is estimated as 191 miles in the reach from Omaha, NE (RM 615.9) to Rulo, NE (RM 498.1). The 191 levee miles may be subdivided as 133.5 miles along the mainstem Missouri River and 57.5 miles of levee tiebacks.

3.5.2. Private Levees. Following levee construction and chute closure, deposited sediment filled many areas riverward of the federal levees. Farming of these areas became extensive. To prevent crop damages caused by normal high flows on the Missouri River, farmers constructed secondary levees at or near the river bank. Many of the secondary private levees tie directly into the federal levees. Private levees have also been constructed along the river bank in areas where federal levees were not constructed. The left bank reach from river mile 515.5 to river mile 498.1 near Rulo, NE is protected solely by private levees.

Based on observations during the 1984 and 1993 floods, secondary and private levees have increased stages. In some cases, the private levees concentrated flows against the federal levees and caused erosion damage. Erosive flow velocities, caused by water flowing though the constrictions of the secondary levee breaches, eroded silts and fine sands from old river chutes leaving large scour holes. Some private levee tiebacks are perpendicular to the federal levee and the flow direction.

Survey data for the private levees is virtually non-existent. Private levee height appears to be near the same height as the federal levee. Many private levees suffered extensive damage during the 1993 event. Field inspection indicates that the majority of the private levees have been repaired (at generally unknown elevation and location). Repair was funded by the SCS and private funds. Repair of private levees has also been funded with PL-8499 funds following previous floods. Total length of private levees along the Missouri River, interior levees, spoil banks, and tiebacks is unknown but is substantial.

3.6. Comparison of 1974 and 1995 Floodplain Sections.

Cross section data available for use with hydraulic modeling was collected in 1974. Since 1974, changes in floodplain elevations have occurred as a result of sediment deposition which reduces capacity and the level of protection provided by levees. Sections were surveyed in 1995 and compared with 1974 data to quantify ground elevation changes which have occurred in the past 20 years. Three separate locations were selected along the Missouri River as test sites. A total of 14 different sections were surveyed in the areas near river mile 513, 531, and 545. Plate OM-2 graphically illustrates comparison at three sections between the original and 1995 surveyed sections. Ground elevations have increased between 1 and 5 feet at most locations surveyed. Comparing the two sections, it also appears that either the original survey did not include enough points to describe private levees constructed along the river bank or that private levee height has significantly increased during the past 20 years.

Brief computations were performed to assess the impact of section aggradation on water surface elevations. The area between the federal levee and the channel conveys a significant amount of flow when

Missouri River channel capacity is exceeded. Variations in roughness due to land use changes also affect capacity but were not assessed by the performed geometry comparison. Computations at each cross section were employed to compare area and conveyance. Section top width was also compared as a check that the two surveys are at the same locations. Results of the comparison are shown in table OM-1.

Table OM-1
Comparison of 1995 and 1974 Floodplain Sections

Missouri River Section	Elev. Range [1]	Top Width Ratio [2]	Area Ratio [2]	Conveyance Ratio [2]	Equiv.[3] Conveyance Elev. Diff.
512.77	859 - 883	0.9919	0.8559	0.7139	3.1
512.97	860 - 883	1.0068	0.9258	0.7481	2.9
513.17	860 - 883	1.0096	0.9023	0.7064	3.4
513.37	860 - 883	0.9923	0.9389	0.7734	2.4
513.58	860 - 883	1.0000	0.9796	0.7924	2.3
530.8	879 - 901	1.0015	0.9023	0.7813	2.0
531.09	879 - 901	0.9921	0.9251	0.8096	1.7
531.32	879 - 901	0.9994	0.8995	0.7739	2.2
531.57	879 - 901	0.9971	0.8897	0.7444	2.4
531.77	879 - 901	0.9929	0.8878	0.7446	2.3
544.88	895 - 917	1.0062	0.9942	0.9221	0.7
545.07	895 - 917	1.0014	0.9855	0.7497	2.4
545.27	895 - 917	1.0079	0.9512	0.7137	2.8
545.78	895 - 917	1.0005	0.8792	0.6297	3.6

[1] Elevation Range defines portion of cross section used for comparison between the approx. normal water surface elev. and top of levee. Section area and conveyance below the low elevation were not included.
[2] Topwidth, area, and conveyance ratios are determined by dividing the new section value by the old section value.
[3] Equivalent Conveyance Elevation Difference refers to the additional elevation required in order for the new section conveyance to equal the old section conveyance.

At each section, an elevation range was selected with the low elevation corresponding to the approximate normal water surface elevation and the high elevation corresponding to near the top of levee elevation. Channel area and conveyance below the normal water surface elevation was not included in the comparison. Area and conveyance ratios were determined by dividing the new section value by the old section value after subtracting the value below the low elevation used at each section. The top width ratio

is nearly 1.0 at all sections which indicates that the two surveys were performed at nearly the same location. The area ratio indicates a decrease of 10 to 15 percent in floodplain area. The conveyance ratio illustrates a larger decrease of 20 to 30 percent. Employing section conveyance to estimate the affect on water surface, the 1974 section conveyed the same amount of flow at an elevation 2 to 3 feet lower than the current section. A complete hydraulic analysis to determine the effect on water surface elevation as a result of the change in section geometry was not performed. However, based on the conveyance comparison, a rough estimate of a 2-3 foot increase appears reasonable.

4. Sediment Effects.

Since construction of federal flood control projects along the Missouri River, significant change has occurred in channel conveyance as a result of aggradation and degradation. Numerous studies have been conducted to quantify Missouri River geometry changes by the Omaha District.

4.1. Gavins Point Dam to Omaha, NE.

Downstream of the Missouri River main stem reservoir system, significant channel degradation has occurred. Degradation analysis and impacts have been outlined in several reports prepared by the Omaha District including Investigation of Impacts to Tributaries from Missouri River Degradation, Volume IV, Supporting Technical Report Missouri River Degradation, and Investigation of Channel Degradation, 1991 Update. Missouri River degradation is a complex issue with several contributing causes. Since construction of Gavins Point Dam (RM 811.05) in 1952, water surface elevations for a discharge of 30,000 cfs have decreased between 4 and 6 feet at Yankton, SD (RM 805.8), Sioux City, IA (RM 732.2), and Decatur, NE (RM 691.0). Many of the tributaries are also experiencing significant degradation. Data analysis indicates that future degradation rates are declining as the river bed becomes more stable. Current data has generally been observed to indicate that Missouri River channel degradation dissipates prior to reaching Omaha, NE (RM 615.9).

4.2. Omaha, NE to Rulo, NE.

An assessment of aggradation and degradation trends was performed in the 1992 Missouri River Channel Capacity Study. The reach between Omaha, NE (RM 615.) and Rulo, NE (RM 498.0) has illustrated general aggradational trends. Since 1955, thalweg elevations have increased by as much as 4 to 6 feet. Based on measured data for a low-flow continuous water surface profile, the increase in water surface elevations varies from 1 to 3 feet. Average bed slope has remained relatively constant at 0.8 to 1.1 foot per mile. An additional increase of 1 - 2 feet in water surface elevation is projected by the year 2020. Within the reach, aggradation at and downstream of the Platte River confluence indicates that the Platte River continues to deliver significant sediment quantities.

4.3. Overbank Sediment Deposition.

Sediment deposition on the overbank or berm area between the channel bank and levee is a common occurrence. Even without a levee, deposition occurs outside of a channel during high flows (mainly during flood recessions) because vegetation traps sediment and increases hydraulic roughness, reducing velocities and sediment transport capacities. A levee project can exacerbate this condition because overbank flows that once spread across a major portion of the floodplain are now confined to a relatively narrow zone adjacent to the river banks. Therefore, a given volume of sediment is deposited over a smaller surface area, resulting in increased deposit depths.

Another general characteristic of this phenomenon is the deposition of the larger size sediment particles immediately adjacent to the channel with a lateral reduction in grainsize down to clay away from the channel. This deposition pattern often results in the formation of natural levees.

4.4. Rating Curve Shifts.

Several studies conducted by the Omaha District have documented substantial shifts in USGS gage rating curves in the last 30-40 years. Shift of the rating curve varies according to location. Within the Missouri River Channel Capacity Study, the rating curve shift from 1952 to 1989, at a discharge of 100,000 cfs, is approximately a rise of 2 feet at Omaha, NE, a 3 foot rise at Nebraska City, NE, and a 3 foot rise at Rulo, NE. Comparison of gage rating curves illustrate a general upward rise at all discharges during the past 30-40 years. The rating curve shift appears to be the largest for flows which exceed 50,000 cfs.

4.5. September 1993 Observations.

A field reconnaissance was conducted in September 1993 to determine the effects of the summer 1993 flooding. Standing water and mud limited this reconnaissance to information that could be gathered near the riverward base of the levee. Material deposited near the levee base consisted entirely of silts and clays. Depths of deposition (determined by digging to the vegetation layer) were on the order of about one foot. Large sand deposits were observed immediately adjacent to the channel. These observations (sand deposits near the channel and silts and clays at a distance away) are consistent with expected floodplain deposition patterns, as discussed in the preceding paragraph.

4.6. April 1994 Observations.

Another reconnaissance was conducted in April 1994. This reconnaissance confirmed the presence of sand deposits immediately adjacent to the channel. It was also noted at this time that lands experiencing the greatest volume of sand deposits were those riverward of failed agricultural levees. These ag levees run parallel to the river and are located inside of the federal tie-back levee. In all likelihood, these levees confined flows, resulting in increased channel velocities, allowing the sand sized particles to be transported through the reach. When they failed, the large concentrations of sand were deposited when the flow spread across the overbank and velocities were reduced.

5. Levee Capacity.

The 1984 Missouri River flood event prompted a study to evaluate the adequacy of the Missouri River levee system from Omaha, NE to Rulo, NE. The discharges used to determine water surface profiles on the main stem were updated from the 1961 study, Missouri River Agricultural Levee Restudy Program. The discharge frequency relationships were updated at the Omaha, NE, Nebraska City, NE and Rulo, NE gaging stations. It was assumed that an unbiased flow regime could be obtained at the gaging stations by subtracting the Gavins Point Dam releases, with an appropriate lag time, from the annual peak flow at each location. After the incremental annual peak flows were determined at each gage, a frequency analysis was performed based on procedures described in Bulletin 17B. The existing discharge capacity of the Missouri River levee units was determined by comparing computed water surface profiles with surveyed levee top elevations. The water surface profiles were computed using an HEC-2 model developed in 1978 for the Special Flood Hazard Information, Missouri River, Gavins Point Dam to Rulo, Nebraska, Volumes I and II. Cross sections and levee top elevations which were used in the HEC-2 model

were surveyed in 1974.

Table OM-2 shows each levee unit, its original design discharge, its capacity to the top of the levee and with two feet of freeboard. Table OM-2 also compares the level of protection each levee provides with two feet of freeboard using 1961 and 1984 hydrology. The last two columns, with 1961 and 1984 hydrology, show the change in level of protection due to discharge. The second and fourth columns show the change in levee capacity due to the factors listed in the paragraph above, such as siltation of the overbanks and tiebacks, etc. It is evident that levee units L-536, R-548, L-550, R-562, R-573, L-575, L-594 and L-601 (all between Nebraska City, NE and Rulo, NE) provide significantly less than 100-year protection to the agricultural land and communities behind them. It should be noted that the hydrology relationships shown in this table are for comparison purposes only. The 1984 study was not approved by the Missouri River Division, so the official hydrology for the Missouri River is that developed in 1961, prior to the main stem reservoir system filling to its normal operating pools. **Note:** The decrease in the levee level of protection shown in table OM-2 was based on hydraulic computations performed with the 1974 survey data and does not include the further capacity reduction as a result of the change in section geometry as documented in table OM-1.

6. 1993 Flood Event.

A thorough discussion of the 1993 flood event is contained within Appendix D of the Post Flood Report (USACE, 1994). The following paragraphs give a brief summary of the description of flooding, levee performance, discharge frequencies and reservoir operations on the mainstem Missouri River from Gavins Point Dam to Rulo, NE.

6.1. Description of Flooding.

The most severe flooding since 1952 occurred on the Missouri River within the Omaha, District from Omaha, NE downstream to the Kansas City District boundary near Rulo, NE. Within this reach record or near record peak discharges were experienced on tributaries to the Missouri River as massive amounts of rainfall fell from July 22-25 over southeast Nebraska and southwest Iowa. Spring snowmelt and earlier rainfall events had left soils saturated and had filled many of the floodplain depressions. Many private and some of the federal levees experienced near levee top stages or were overtopped. Private levee failures were common, but the federal levees performed remarkably well with only one complete failure. No federal levee failures were due to overtopping. Many levees that did not fail experienced significant damage. Agricultural damages were extensive from both river flooding and inadequate or inoperative interior drainage facilities. Many smaller communities and towns also suffered extensive damage to homes, businesses and infrastructure.

6.2. Levee Performance.

Overall, the federally constructed levees performed very well. As a result of the extremely high flows, all federal levees from unit L-575 downstream to unit R-520 experienced some overtopping either on the mainstem or a tieback levee. Overtopping was generally over a short levee section with limited depth and duration. Levee unit L-550 was overtopped for approximately 10 hours in the area north of the Brownville, NE bridge, during the evening of 23 July and the early morning of 24 July. Based on accumulated debris, overtopping depth was estimated from 0 to 2.0 feet. The morning of 24 July, the sustained high water initiated a geotechnical failure of the levee. The failure mechanism was either underseepage or embankment seepage related. The peak stage at the breach location was approximately

Table OM-2 CAPACITY OF MISSOURI RIVER LEVEE UNITS					
				Degree of Protection With 2 feet of Freeboard	
Levee Unit (Yr Cmpltd)	Design Discharge (cfs)	Discharge Capacity to Top of Levee (cfs)	Discharge Capacity with 2 ft of Freeboard (cfs)	With 1961 Hydrology (year)	With 1984 Hydrology[3] (year)
R-520 (1960)	310,000	410,000	340,000	500+	125
L-536 (1951)	306,000	316,000	250,000	200	40
L-550 (1951)	305,000	252,000	204,000	50	20
R-548 (1951)	304,000	251,000	206,000	50	20
R-562 (1949)	300,000	248,000	201,000	70	30
L-575 (1949)	295,000	268,000	220,000	100	40
R-573 (1949)	295,000	279,000	200,000	70	30
L-594 (1964)	295,000	312,000	242,000	250	70
L-601 (1966)	295,000	267,000	226,000	150	50
[1] L-611-614 (1986)	295,000	313,000	295,000	500	200
[2] L-611-614 (1986)	250,000	268,000	250,000	500	500
R-613 (1971)	250,000	303,000	240,000	400	400
R-616 (1986)	250,000	313,000	250,000	500	500
L-624 (1950)	250,000	303,000	256,000	500+	500+
L-627 (1950)	250,000	345,000	297,000	500+	500+
Council Bluffs (1950)	250,000	298,000	264,000	500+	500+
Omaha (1950)	250,000	298,000	264,000	500+	500+

NOTE: This table was taken from Adequacy of the Missouri Levee System from Rulo to Omaha, completed in April 1986. Levee capacity values employed 1974 survey data and do not include results of cross section comparison performed in 1995.
[1] Represents the portion of levee L-611-614 downstream of the Platte River.
[2] Represents the portion of levee L-611-614 upstream of the Platte River.
[3] Values not officially adopted or employed.

3-4 feet below the levee crest. An extensive scour hole developed at the breach location. Depth of the scour hole was approximately 25-30 feet. The extensive scour depth area extended to either side of the

levee a distance of 500-1000 feet. Breach width was approximately 300 feet. Breach width did not increase from the initial failure while flow was passing through the breach. Based on rough staff gages, the interior ponding area filled within 24 - 36 hours. A second breach formed at the downstream end of the levee, near the Rock Creek tieback at RM 525, from upper breach flows reentering the Missouri River. At this location, the breach width was near 300 feet with a maximum scour depth of 30 - 40 feet.

6.3. Discharge Frequency.

Peak discharges on the Missouri River for 1993 varied from 28,500 cfs at Yankton, SD (RM 805.8), to 115,000 cfs at Omaha, NE (RM 615.9), and 307,000 cfs at Rulo, NE (RM 498.0). Based on discharge frequencies developed for the 1986 Adequacy of Missouri River Levee System report, the return period varied widely from a 2-5 year event at Omaha, NE to a 50-100 year event at Rulo, NE. A summary of peak discharge, estimated frequency, and gage data is illustrated in Table OM-3. As the table illustrates, major inflow to the Missouri River occurred downstream of Omaha, NE from the tributaries within the reach.

6.4. Missouri River Reservoir Operation.

The six main stem reservoirs had a significant impact on reducing the peak stage experienced along the Missouri River downstream from Gavins Point Dam. It is estimated that the total main stem storage increased by more than 9 million acre-feet (maf) from June through August 1993. When combined with the 61 tributary reservoirs located within the Missouri River Division, it is estimated that a total of more than 16 maf of water was stored by the first of August. Without the main stem reservoirs, the duration of flooding would have increased from about 0 to 60 days at Sioux City, IA, from 1 to 67 days at Omaha, NE and from 25 to 80 days at Nebraska City, NE.

7. Hydraulic Unsteady Flow Model.

An unsteady flow model was constructed for purposes of the FPMA by the Omaha District. The unsteady flow model employed geometry data collected during previous studies. UNET was selected as the type of unsteady flow model. Unsteady flow model construction required coordination and interaction between Corps District offices responsible for modeling of their reaches of the Missouri and\or Mississippi Rivers and tributaries.

7.1. Previous Modeling Studies.

The last major modeling studies developed for the mainstem Missouri River from Gavins Point Dam to Rulo, NE, consisted of a steady state HEC-2 backwater model for the 1978 Special Flood Hazard Information, Missouri River, Gavins Point to Rulo, Nebraska and an unsteady flow UNET (HEC,1992) model for the 1992 Emergency System Operating Plan (ESOP), Missouri River, Main Stem Reservoir System, Gavins Point to Rulo, Nebraska. The ESOP model was determined to be unusable for several reasons. This includes, using only 57 cross sections for the Missouri River from Gavins Point Dam to Rulo, NE, not modeling any of the major tributaries as routing reaches, not utilizing the available calibration tools found in UNET and a very poor calibration to daily discharge values for the 1984 flood event on the Missouri River. The calibration was deemed poor based on a modeling alternative in the ESOP study that showed that a steady flow of 600,000 cfs could pass by without overtopping Federal Levee L-550. In 1993, with a flow of somewhere between 200,000 cfs and 300,000 cfs, L-550 was overtopped.

Table OM-3 Missouri River Mainstem and Tributaries Peak Discharge and Gage Height Summary								
STATION	PERIOD OF RECORD	PEAK OF RECORD			WY 1993 PEAK			Estimated Frequency in Years
		DATE	gh	c.f.s.	DATE	gh	c.f.s.	
SOUTH DAKOTA								
Missouri River at Yankton	1931-1993	4/13/52	35.50	480,000	Oct 9	15.35	28,500	25-50
James River nr Yankton	1982-1993	6/23/84	24.34	26,400	Jul 8	21.15	15,800	5-10
Vermillion River nr Vermillion	1984-1993	6/23/84	31.77	21,400	Jul 8	26.68	10,200 BW	ID
IOWA								
Big Sioux River at Akron	1929-1993	4/9/69	22.99	80,800	May 10	23.05	66,700	25-50
Missouri River at Sioux city	1938-1993	4/24/52	24.28	444,100	Jul 15	27.33	72,200	2-5
Floyd River at James	1935-1993	6/8/53	25.30	71,500	Mar 29	22.34	9,680	5-10
Monona-Harrison Ditch nr Turin	1939-1993 b	2/19/71	28.03	19,900	Mar 27	20.48	6,980	2-5
Little Sioux River nr Turin	1958-1993	6/21/83	26.54	31,200	Jul 19	23.96	26,300	5-10
Soldier River at Pisgah	1941-1993	6/12/50	28.17	22,500	Jul 9	27.56	23,400	10-25
Boyer River at Logan	1938-1993	6/17/90	22.54	30,800	Jul 9	22.19	26,300	10-25
Nishnabotna River above Hamburg	1929-1993	6/24/47	26.03	55,500	Jul 25	30.56	37,700	25-50
NEBRASKA								
Bow Cr. nr St. James	1979-1993	6/21/84	13.23	21,400	Mar 8	7.80	5,610	2-5
Omaha Cr. at Homer	1946-1993	2/19/71	28.47	18,100	Mar 7	11.56	ice	2-5
					Jul 9	11.36	6,820	
Missouri River at Omaha	1929-1993	4/18/52	40.20	396,000	Jul 10	30.26	115,000	2-5
Platte River at Louisville	1954-1993	6/14/84	11.34	144,000	Jul 25	11.90	160,000	25-50
Weeping Water Cr. at Union	1951-1993	5/9/50	29.80	60,300	Jul 23	30.97	65,100	50-100
Missouri River at Nebraska City	1930-1993	4/19/52	27.66	414,000	Jul 23	27.19	196,000	10-25
Little Nemaha River at Auburn	1950-1993	5/9/50	27.65	164,000	Jul 24	26.49	105,000	25-50
Missouri River at Rulo	1950-1993	4/22/52	25.60	358,000	Jul 24	25.37	307,000	50-100

b - Broken Record d - Discontinued Record e - Estimate n - New Datum
NA - Not Available BW - Back Water ID - Insufficient Data

7.2. Unsteady Flow Model - UNET.

UNET was selected as the unsteady flow model for the basin wide modeling tool. UNET is a one-dimensional unsteady flow program developed by Dr. Robert L. Barkau, which includes the capability of simulating a complex network of open channels. Unsteady flow routing accounts for the variation in flow with both time and space. UNET is considered a complete dynamic wave model since it solves the full St. Venants equations of momentum and continuity equations. The dynamic wave routing method was used for this analysis because it has the ability to account for critical backwater effects in the routing and can directly simulate flows that spill over or breach a levee and pond behind the levee in predefined storage areas. Also, UNET allows for the use of observed streamflow hydrographs for upstream boundary conditions as well as lateral tributary inflows.

UNET model input consists of three parts, the geometry file (csect), boundary conditions file (bc), and HECDSS hydrograph data files. The csect input file consists of the HEC-2 style cross sectional geometry developed by the user. The geometry is then converted into tables of elevation versus area, conveyance, and storage. Csect also tabulates interior boundary conditions and resolves network connections between reaches and storage areas. The bc file includes the upstream and downstream boundary conditions, time windows for simulation, computational time step, HECDSS inflow hydrograph pathnames and calibration factors. The HECDSS database file stores all the stage and flow hydrographs read by the UNET program for boundary conditions and lateral inflow hydrographs. Also, the UNET program writes the following output data to DSS: 1)discharge and stage hydrographs, 2) maximum water surface profiles, 3) instantaneous discharge and stage profiles 4) channel invert and bank profiles, and 5) elevation versus conveyance and area properties.

For purposes of the FPMA, the UNET program was further developed by Dr. Robert Barkau. Program changes include enhancements to the levee failure algorithm, calibration methods, and conveyance within the overbank.

8. UNET Model Development.

A UNET model of the Missouri River was developed by the Omaha District to be used for the FPMA. Model limits extend from downstream of Gavins Point Dam at river mile 811.05 to below Rulo, NE, at river mile 498.0. The Omaha District Missouri River reach was divided into 2 separate UNET models. The upper model extends from Gavins Point Dam (RM 811.05) downstream to Omaha, NE (RM 615.97). The lower model extends from Omaha, NE, (RM 615.97) downstream to below Rulo, NE (RM 498.0). The model separation point corresponds with the location of the Missouri River USGS gaging station located at the Interstate 480 bridge. For purposes of the FPMA, the majority of the UNET modeling concentrated on the lower portion of the Missouri River downstream of Omaha, NE. Federally constructed levees along the Missouri are not present upstream of river mile 624.9. The lower model from Omaha, NE to Rulo, NE uses the observed stage hydrograph at the USGS gage or results of the upper model as the upstream boundary condition. The UNET model is broken into a series of routing reaches along the Missouri River and major tributaries. The lower Missouri River UNET model includes federal levees, storage areas, and model overflow during major events. A summary of the routing reaches is shown in Table OM-4. A schematic of the Missouri River, significant tributaries, and levee cells included within the UNET model is illustrated in plate OM-3.

8.1. Geometry.

Table OM-4 Missouri River UNET Model Reaches Gavins Point to D/S Rulo			
Reach No.	**River**	**Location**	**Reach Length**
		Upper Missouri River UNET Model	
1	Missouri River	Gavins Point to Big Sioux River	RM 811.05 to 734.23
2	Big Sioux River	Akron, IA Gage to Missouri River	Approx. 35 Miles
3	Missouri River	Big Sioux River to Little Sioux River	RM 734.23 to 669.17
4	Little Sioux River	Turin, IA Gage to Missouri River	Approx. 13.5 Miles
5	Missouri River	Little Sioux River to Soldier River	RM 669.17 to 663.97
6	Soldier River	Pisgah, IA Gage to Missouri River	Approx. 13.1 Miles
7	Missouri River	Soldier River to Boyer River	RM 663.97 to 635.21
8	Boyer River	Logan, IA Gage to Missouri River	Approx. 15.8 Miles
9	Missouri River	Boyer River to Downstream of Omaha	RM 635.21 to 600.0
		Lower Missouri River UNET Model	
1	Missouri River	Omaha U/S Boundary to Platte River	RM 615.97 to 594.76
2	Platte River	Louisville, NE Gage to Missouri River	Approx. 16.5 Miles
3	Missouri River	Platte River to Nishnabotna River	RM 594.76 to 568.63
4	Weeping Water Cr	Union, NE Gage to Missouri River	Approx. 6.3 Miles
5	Missouri River	Weeping Water Cr. to Nishnabotna River	RM 568.63 to 542.01
6	Nishnabotna River	Hamburg, IA Gage to Missouri River	Approx. 13.7 Miles
7	Missouri River	Nishnabotna River to Little Nemaha River	RM 542.01 to 527.79
8	Little Nemaha Riv	Auburn, NE Gage to Missouri River	Approx. 10.3 Miles
9	Missouri River	Little Nemaha River to Big Nemaha River	RM 527.79 to 494.83
10	Big Nemaha River	Fall City, NE Gage to Missouri River	Approx. 14.5 Miles
11	Missouri River	Big Nemaha River To D/S Boundary Below Rulo, NE	RM 494.83 to 410.0

Note: Upper and Lower Missouri River UNET model refers to the two separate models developed by the Omaha District for the FPMA

8.1.1. Mainstem Missouri River.

Cross section data for the Missouri River was compiled from existing HEC-2 data. The HEC-2 models were developed in 1978 for the Special Flood Hazard Information, Missouri River, Gavins Point Dam to Rulo, Nebraska, Volumes I and II. Cross sections and levee top elevations utilized in the model were surveyed in 1974 and earlier. Significant changes to the channel and overbank area may have occurred since collection of the data as outlined in paragraph 3.6. Cross section interval on the Missouri River is roughly 1000 feet. There are approximately 1,750 cross sections total in the two UNET models.

The cross section data from the HEC-2 models had to be revised in several ways to be compatible for UNET. This included reducing the number points used to define the cross section to 100 or less, converting the NH record roughness values from a lookup table to a left overbank, right overbank and channel roughness value on the NC record, and incorporating ET record stations with the X3 record. Bank stations were moved to the correct location for use with the UNET model. Bridge cross sections were reviewed and revised to correctly reflect flow restrictions for UNET. All bridge deck records (BT) were removed as these were not impacted by 1993 flood levels. Existing X3 record stationing and elevations were revised to reflect levee locations or effective flow areas modeled and to correctly reflect the storage-conveyance relationship.

Initial HEC-2 backwater runs were performed and computed stages compared with gaging station rating curves, highwater mark data and measured flows. Cross sections were then reversed for the UNET model. HEC-2 requires cross sections to be in a downstream to upstream format while UNET requires a upstream to downstream format.

In order to provide a stable boundary at Rulo, NE and compute reasonable results, a significant reach downstream of Rulo, NE is required. The lower Missouri River UNET model employed for all simulations extends to downstream of St. Joseph, MO at RM 448.2. The Missouri River Omaha District Civil Works boundary corresponds with Rulo, NE at RM 498.0. Geometry data for downstream of Rulo, NE, was developed and provided to Omaha District by Kansas City District. Kansas City District also provided information regarding levee cells within the reach downstream of Rulo, NE.

8.1.2. Tributaries.

Routing of the major tributaries from the gaging station location to the confluence with the Missouri River was determined to increase model accuracy. Routing of tributary flows changes timing which can be significant for large peak inflows. Timing of tributary inflows is critical for reproducing computed peak stages and discharges. Unfortunately, surveyed geometry data was available for only one tributary.

Cross section geometry for significant tributaries was coded for all major tributaries for the reach from the confluence with the Missouri River upstream to the USGS gaging station location. Most tributary gaging stations are located approximately 10-15 river miles upstream of the confluence with the Missouri River. Tributary cross section data were taken from USGS 7.5 minute quadrangle topographic maps or the best available topographic information. Tributary cross section interval varied from 5,000 to 20,000 feet. As a result of the poor cross section data, computed stage information on the tributaries is not accurate. Tributary cross sections for the Nishnabotna River were developed from available HEC-2 models.

A rough HEC-2 model was constructed and calibrated from the tributary gage rating curve. As a final step, the tributary HEC-2 models were reversed for use with the UNET model csect input file. Tributary modeling efforts were of limited detail and intended for flow routing only. Computed stage information on the tributaries had little or no value.

8.2. Roughness.

The original HEC-2 models employed NH records at each cross section. Model simulations which employed the original NH roughness values within the UNET model did not compare very well with

observed data. In order to reduce model complexity while not affecting model accuracy, all NH records were removed and replaced with NC records. For describing Missouri River channel roughness, average Mannings' n values employed within the model ranged from 0.019 to 0.023. Within the overbank, average Mannings' n values ranged from 0.045 to 0.060.

8.3. Boundary Conditions.

The upper Missouri River model upstream boundary uses the flow hydrograph at Yankton, SD. With the exception of the Platte River, tributaries which are included as routing reaches employ the USGS gaging station flow hydrograph as the upstream boundary. Within the lower Missouri River model, upstream boundary conditions at Omaha, NE and for the Platte River routing reach employed a stage boundary. A stage boundary condition allows the model to compute inflow based on geometry. For model simulations which required employing both the upper and lower Missouri River UNET models, the lower model employed the hydrograph computed by the upper UNET model at the Omaha, NE location. Within the lower Missouri River UNET model, the location of the upstream boundary stage hydrograph corresponds with the USGS gaging station at Omaha NE (RM 615.97).

Both models utilize a Mannings n relationship for the downstream boundary condition. For each of the UNET models, the downstream model accuracy limit corresponds to the gaging station located at Omaha, NE and Rulo, NE for the upper and lower models, respectively. Within each of the UNET models, further cross sections are included below the downstream gaging station location for a distance of 20-30 river miles. Additional cross sections are required to reduce the effect on computations of the artificial Mannings n downstream boundary condition. Model computations in the vicinity of the boundary condition are influenced by the assumed boundary and not reflective of river flow characteristics.

8.4. Model Time Step.

A maximum UNET model time step of 3 hours was assumed in order to reduce model run time. Attempts at modeling the Rulo overbank area as a conveyance reach with levee overtopping from the Missouri River required a time step of 5 minutes for model stability. A 5 minute time step was not conducive to performing model computations for the base condition and all the various alternatives. Comparison of model results computed with various time steps was performed. A 3 hour time step produced results which did not deviate from simulations employing shorter time steps. Model accuracy was not affected by further reducing the time step.

8.5. Hydrologic Data.

Discharge and stage hydrographs for the Missouri River and tributaries were required for the UNET model as input, boundary conditions, calibration, and verification. The hydrologic data was taken from the USGS' Automated Data Processing System (ADAPS) which is part of the National Water Information System (NWIS). Provisional discharge and stage data were available along with rating tables and actual discharge measurements for all USGS gages.

For the UNET model, hourly discharges and stage data for the period June 1993 through September 1993 were obtained for USGS gages along the mainstem Missouri River and its tributaries from Yankton, SD to St. Joseph, MO. Also, to aid in the calibration of the UNET model, the Corps of Engineers has stage gages on the Missouri River at Plattsmouth, NE and Brownville, NE. Stage data were collected from the COE gages for the same time period. Actual discharge measurements and rating tables were gathered

for the mainstem Missouri River gages and for the Platte River gage at Louisville, NE. USGS and COE streamflow gages with their locations, gage identification numbers and other pertinent data are shown in Table OM-5 for the tributaries and Table OM-6 mainstem Missouri River.

Due to the large amount of hydrologic data required for the model, UNET is coupled with the Hydrologic Engineering Center's data storage system HECDSS (HEC,1990). The discharge and stage hydrograph and actual discharge measurements were imported into HECDSS. Stage hydrographs were converted to the 1929 National Geodetic Vertical Datum (NGVD) using DSSMATH (HEC,1992). Bad and missing data were interpolated and replaced. Data were then read directly into the UNET model.

Table OM-5
Missouri River UNET Model Inflow Data
Gavins Point to St. Joseph, MO

Tributary Gage and Location	USGS Gage ID	Missouri River Location	UNET Model Input Method
Upper Missouri River UNET Model			
James River at Scotland, SD	06478500	RM 797.7	Lateral Inflow
Bow Creek nr St. James, NE	06478518	RM 787.8	Lateral Inflow
Vermillion River nr Vermillion, SD	06479010	RM 772.2	Lateral Inflow
Big Sioux River at Akron, IA	06485500	RM 734.2	Routing Reach
Perry Creek at Sioux City, IA	06600000	RM 732.1	Lateral Inflow
Floyd River at James, IA	06600500	RM 731.3	Lateral Inflow
Omaha Creek at Homer, NE	06601000	RM 719.8	Lateral Inflow
Monona Harrison Ditch at Turin, IA	06602400	RM 670.0	Lateral Inflow
Little Sioux River nr Turin, IA	06607500	RM 669.2	Routing Reach
Soldier River at Pisgah, IA	06608500	RM 664.0	Routing Reach
Boyer River at Logan, IA	06609500	RM 635.2	Routing Reach
Lower Missouri River UNET Model			
Papillion Creek at Fort Crook, NE	Corps Gage	RM 596.6	Lateral Inflow
Platte River at Louisville, NE	06805500	RM 594.8	Routing Reach
Weeping Water Creek at Union, NE	06806500	RM 568.6	Routing Reach
Nishnabotna River above Hamburg, IA	06810000	RM 542.0	Routing Reach
Little Nemaha River at Auburn, NE	06811500	RM 527.8	Routing Reach
Big Nemaha River at Fall City, NE	06815000	RM 494.8	Routing Reach
Nodaway River at Graham, MO	06817700	RM 463.0	Routing Reach

8.6. Ungaged Inflows.

Of the 135,400 square mile incremental Missouri River drainage area between Gavins Point Dam and Rulo, NE, there is approximately 7,000 square miles that is ungaged. Between Omaha, NE and Rulo, NE, the runoff of the ungaged areas became significant due to rainfall amounts of up to 18 inches during the month of July, 1993. Therefore, an estimate of the runoff for the ungaged area between Omaha, NE and Rulo, NE was made using the index drainage basin method as described below.

Table OM-6
Missouri River Gaging Station Data
Gavins Pt to St. Joseph, MO

Missouri River Gage Location	USGS Gage ID	River Mile Location
Yankton, SD - 5.2 Miles D/S of Gavins Point Dam	06467500	RM 805.8
Sioux City, IA - 1.9 Miles D/S of Big Sioux River	06486000	RM 732.2
Decatur, NE - 0.1 Miles U/S of Hwy 175	06601200	RM 691.0
Blair, NE	Corps Stage Gage	RM 648.3
Omaha, NE - 0.1 Miles D/S of I-480	06610000	RM 615.9
Plattsmouth, NE - 3.2 Miles D/S of Platte River	Corps Stage Gage	RM 591.5
Nebraska City, NE - 2.0 Miles U/S of Hwy 2	06807000	RM 562.6
Brownville, NE - 6.8 Miles D/S of Nishnabotna River, 7.4 Miles U/S of Little Nemaha River	Corps Stage Gage	RM 535.2
Rulo, NE - D/S Hwy 159 and 3.2 Miles U/S of Big Nemaha River	06813500	RM 498.0
St. Joseph, MO	06818000	RM 448.2

For determining the streamflow for ungaged areas, several streamflow gages were used as index gages, The East Nishnabotna River at Red Oak, IA, North Fork Big Nemaha River at Humboldt, NE and Weeping Water Creek at Union, NE. The areas for the ungaged were planimetered from USGS 1:250,000 scale topographic maps. Ungaged drainage basin areas were then divided by the index gage drainage basin area to develop an index ratio. Streamflows for the index gage were then multiplied by the index ratio to come up with a discharge hydrograph for the ungaged area. In the UNET model the inflow hydrographs for the ungaged tributaries were treated as lateral inflows or uniform lateral inflows. The ungaged tributaries, locations, index gages and index ratios are shown in Table OM-7.

8.7. Levee and Storage Area Data.

Within the Omaha District, areas to the landward side of the federal levees were modeled as storage areas. Large overbank areas behind the federal levees will affect model timing and computed results if the storage cells have a significant amount of flow through them. Levee overflow from the main channel into the adjacent overbank requires additional coding of data within the UNET model.

TABLE OM-7 Ungaged Tributary Location and Index Gage Missouri River from Omaha to Rulo, NE				
Ungaged Tributary	**Location**	**River Mile**	**Drainage Area (Sq. Mi.)**	**Ratio w/ Index Gage**
Index Gage: East Nishnabotna River at Red Oak, IA - D.A. = 894 Square Miles				
Tarkio River	Missouri River	RM 507.6	508	0.22
Index Gage: North Fork Big Nemaha River at Humboldt, NE - D.A. = 548 Square Miles				
Muddy Creek	Big Nemaha River	Station 30000	258	0.47
Index Gage: Weeping Water Creek at Union, NE - D.A. = 241 Square Miles				
Platte River Tributary	Platte River	Station 24100	105	0.44
Waubonsie Creek	Missouri River	RM 581.1	36	0.15
Plum Creek	Missouri River	RM 573.1	36	0.15
Numerous Small Tributaries Neb. City to Nishnabotna River	Missouri River	RM 562.6 - 542.0	53	0.22
Nishnabotna Tributary	Nishnabotna River	Station 20200	62	0.26
Numerous Small Tributaries Nishnabotna River To Brownville	Missouri River	RM 542.0 - 535.2	28	0.12

In order to accurately model overflow into leveed cells, overbank areas were subdivided by their federal levee designation. The larger Federal levees, L-575 and L-550, were subdivided into two smaller storage areas each using Iowa Highway 2 and U.S. Highway 136, respectively. Surface areas were digitized from USGS 1:250,000 scale topographic maps. Storage cell invert elevations were based on the average ground elevation behind the levees. Topographic features such as levees, roads and railroads were utilized to subdivide the storage cells. The SC record for the csect file in UNET was used to interconnect adjacent storage areas based on topography and flow structures. Plate OM-3 shows the general Missouri River mile location and size of the storage areas.

For each storage cell, an existing top of levee elevation and location was required to model an overflow/breach. For the 1993 base condition, the actual levee overflow/breach locations were utilized in the UNET model. For alternative conditions which affected levee overflow, potential overflow locations were added to the model. As a minimum for each cell, overflow locations were specified at the upstream

and downstream end. The distance between overflow locations varied from 5 to 10 river miles.

8.8. Levee Failures and Overtopping.

The simulation of levee failures during the 1993 event was performed utilizing the simple embankment failure record, SF found in the csect file of UNET. Simulation of levee failure requires the collection of data for levee failure mechanisms, breach section geometry, areas inundated, and time to fill the inundated areas. Data from the 1993 event for L-550 indicated that the levee breach formed at a rate such that the interior storage area filled within 24 to 36 hours. Determination of additional levee failure parameters was performed during calibration.

Federal levee L-550 and several other federal levees also experienced overtopping during the 1993 event for an average period of 6-12 hours. However, the UNET model was not configured to account for the overtopping because the volume of flow overtopping the levee was small compared to the amount of flow going through the levee breach.

8.9. Rulo Overflow Reach.

The Rulo overflow reach refers to the section of the Missouri River from river mile 482.8 to 515.5. Within this reach, the left bank is protected by a series of private levees. The level of protection from the private levees was estimated as 10 to 20 year. Floodplain width from the Missouri River to the bluff along the left bank varies from 3-9 miles. The floodplain contains the Squaw Creek National Wildlife Refuge and the Big Lake recreational area. A Missouri River USGS gaging station is installed on the U.S. Highway 159 bridge at Rulo, NE (RM 498.0). During the 1993 event, the private levees in the Rulo, NE reach experienced considerable damage with numerous breaches. A significant amount of flow occurred in the overbank area. For the estimated peak discharge measurement of approximately 300,000 cfs, estimates are that approximately 30-40% of the flow bypassed the Rulo, NE gaging station. In order to simulate the large amount of flow in the overbank, an additional conveyance reach was included in the UNET model for the left overbank. A series of artificial storage cells were constructed between the Missouri River channel reach and the Rulo overbank reach in order to increase UNET model stability. Numerous connections between the storage cells and adjacent reaches were established. Also for reasons of model stability, the constructed Rulo overbank reach was a prismatic channel and did not resemble actual geometry.

8.10. Calibration Event.

Calibration of the two UNET models was performed for the Gavins Point Dam to Omaha, NE and Omaha, NE to Rulo, NE reaches. Calibration efforts employed discharge/conveyance relationships on a reach by reach basis. Model calibration centered on the flood event from June through August 1993. The 1984 flood event on the Missouri River would also have been ideal for calibration because it was unaffected by levee overtopping or failure. However, only daily flows and stages were readily available for the USGS streamflow gages and the COE's stage gages at Plattsmouth and Brownville, NE were not yet functioning. Due to the lack of inflow data, simulation of the 1984 event was not performed.

8.11. Initial Model Testing.

Following assembly of the csect and boundary condition files, initial model testing was performed to discover geometry errors, model inconsistencies, and assess model accuracy. Initial simulations were

performed with a range of constant discharges. Comparison between the UNET and HEC-2 profile for computed stages was performed. Simulations were also performed with a complete flow hydrograph in order to assess model stability.

9. UNET Model Calibration.

Calibration of the UNET model was an iterative process performed in several stages. Calibration efforts focused on reproducing observed stage hydrographs at gaging stations along the Missouri River and verifying with discharge measurements. Initial calibration efforts employed the NC record for channel and overbank roughness within the csect file. Once the model was accurate to within 2-3 feet, additional calibration was performed using conveyance change and discharge-conveyance relationships within the bc file for separate reaches within the model. The conveyance change relationship applies a constant factor to the cross section conveyance and storage determined by csect. The discharge-conveyance relationship applies a factor to cross section conveyance which may be varied according to flow rate. Final calibration is a combination of the effects of all the parameters employed in both the csect and bc files.

9.1. Model Stage Calibration. Calibration of computed model stages was performed employing stage hydrographs at Missouri River gaging stations located at Omaha, NE, Plattsmouth, NE, Nebraska City, NE, Brownville, NE, Rulo, NE and St. Joseph, MO. Calibration was considered complete when computed model stages were within 0.5 feet of observed stages. Calibration of tributary routing reaches was not performed. Stage calibration was performed on a system wide basis for the entire hydrograph. However, the calibration effort focused on reproducing peak stages. Calibration of peak stages at the Rulo, NE gage were particularly difficult. Measured data indicated that 30 - 40 percent of the peak flow rate was in the overbank flow area. Numerous attempts were required to provide reasonable results at Rulo, NE and within both upstream and downstream locations. Modeling methods at Rulo, NE attempted to account for the complex flow phenomena which occurred with simplified techniques. Plates OM-4 through OM-8 provide a comparison of computed stages and observed stages.

9.2. Model Flow Issues.

All inflow to the UNET model was obtained from USGS gaging station records. With the exception of measured flow adjustments, gaging station discharges are generally determined from a stage - discharge rating curve. Plots of 1993 measured flow data versus gage rating curves were developed at Omaha, NE, Nebraska City, NE, and Rulo, NE on the Missouri River and Louisville, NE on the Platte River. Plotted data is shown in plates OM-9 through OM-12. As the plates illustrate, significant flow shifts occurred at all of the gaging stations during the 1993 event. For these reasons, measured flow data was employed for purposes of model discharge calibration instead of gage rating curves.

Following initial calibration attempts, the variation in model results indicated that flow within the model required adjustment. Comparing the volume of flow at the Nebraska City, NE Gage with volume at the Omaha, NE gage and tributary inflow gages between the two Missouri River gages resulted in a volume decrease at Nebraska City, NE. The decrease in volume is magnified if ungaged inflows to the reach are also included. Peak Platte River discharges during the 1993 flood event, which enters the Missouri River between Omaha, NE and Nebraska City, NE, also appeared to be extremely high.

In order to provide more reasonable results, upstream boundary conditions for the Platte River at Louisville, NE and the Missouri River at Omaha, NE were converted from flow to stage. Employing a

stage boundary condition allows the UNET model to determine inflow from the observed stage hydrographs for the Platte and Missouri Rivers. Flow corrections were then made to include ungaged drainage areas. Accounting for ungaged inflows and correctly incorporating gage data was a major challenge in the development of an accurate UNET model.

9.3. Model Flow Calibration.

While the UNET model was calibrated to observed stage hydrographs, discharges still need to be verified to assure model accuracy. But, as discussed in the preceding paragraph, problems were encountered using USGS discharge hydrographs that are based on a rating curve and shifts to the rating curve based on a single discharge measurement. This was evident at the Louisville, NE gage on the Platte River where flows appeared to be overestimated. For these reasons, observed discharge hydrographs could not be used to verify the UNET model. In order to assess model accuracy, the computed discharge hydrographs from the UNET model were compared to actual USGS discharge measurements taken at the gaging stations. Discharge measurements are taken at least once a week on the mainstem Missouri River. Plates OM-13 through OM-17 compares the computed model discharges to the discharge measurements at the gaging stations. The results show that the computed discharges for the UNET model were nearly the same as the discharge measurements.

9.4. Calibration Results.

Final parameters in the calibrated UNET model included a time step of 3.0 hours, Mannings' n values ranging from 0.019 to 0.023 for the channel and 0.045 to 0.060 for the overbank, and a theta value of 0.6. Conveyance change calibration factors were generally in the range of 0.8 to 1.2 throughout the model. Theta was changed to 1.0 to assure model stability and consistent results when the UNET model was configured for other alternatives.

Following final calibration of the UNET model, water surface elevations were compared with available highwater mark data to verify the calibrated roughness values. Computed stages were generally within 1-2 feet of highwater mark data. A comparison of peak discharges shows that the UNET model is within five percent or less of the observed values for the gaging stations on the Missouri River. Comparison of peak discharges at Rulo, NE was not possible due to way the Rulo overbank was modeled in UNET. Discharges in the Rulo overbank could not be readily combined with the flow in the channel. Computed peak stages for the UNET model were all within +/- 0.3 feet or less of the observed stages at the Missouri River gaging stations, except the Yankton, SD gage which was -2.4 feet. Based on comparison with measured flow data at St. Joseph, MO, the method of modeling the Rulo overbank area with a separate routing reach appears to have successfully conveyed flow through the overbank area with the proper timing and attenuation. Comparison with the measured discharges at Louisville, NE and Nebraska City, NE verify model accuracy at the stage boundary condition for the Platte River.

10. Alternatives.

Several alternatives were analyzed to address flood plain conditions and study objectives as outlined in the correspondence authorizing the study. This included various alternatives involving the existing agricultural levees and several upland retention/watershed measures both structural and nonstructural. Agricultural levee alternatives include levee removal, levee confinement to contain the 1993 event and altering levees to provide only a 25-year level of protection. Upland retention/watershed measures include no mainstem Missouri River reservoirs, runoff reductions of 5% and 10%, and revised

operation of mainstem Missouri River reservoirs. All alternatives were system wide and included passing flow and stage information from upstream districts to downstream districts. One case study that involved just the Omaha District UNET model was a levee setback alternative. All alternatives are discussed in detail in the following paragraphs.

11. Agricultural Levee Alternatives.

The effects of several alternative agricultural levee height and locations were analyzed employing the calibrated UNET model developed for the base condition. For each alternative, the base condition UNET model was modified to reflect geometry changes required to simulate the effect on conveyance within the model. Calibration parameters determined in the base condition were not altered for any of the alternatives. In reality, the alternatives alter conveyance within a cross section by changing effective flow area, land use, sediment deposition, and other factors. Since no federal agricultural levees exist upstream of Omaha, NE, only the lower Missouri River UNET model was employed to assess the alternatives within the Omaha District.

11.1. Levee Removal.

For this alternative, all agricultural levees were removed. Simulations were performed with both a minimum and maximum roughness level within the overbank area. Roughness values of 0.08 and 0.32 were selected to provide a reasonable lower and upper bound for computed results. Factors affecting conveyance were not evaluated in detail. For example, removal of the levee would not result in an effective flow width equal to the entire valley width. Physical factors such as channel meandering, vegetation, topography, structures such as roads and railroads, and other components will restrict effective flow width to a value much less than the cross section width. Various forms of land use within the overbank such as farming and natural habitat will have considerably different roughness values. Levee removal will remove channel constraints such that channel meandering and overbank sediment deposition may actually reduce conveyance.

11.1.1. UNET Model Modifications. Modifying the UNET model to accurately reflect the conveyance changes at every cross section was not feasible. The selection of an upper and lower roughness value was performed to encompass all the various elements which affect conveyance within a single parameter. Roughness values were changed for all overbank areas. Channel roughness values were not modified. If levees were removed and the channel was no longer maintained for navigation, channel roughness values may increase as the cross section adapts. Levee removal was simulated by removing the encroachment station (delete the X3 record) from each cross section.

11.1.2. Model Cautions. Removing the X3 records results in employing the entire cross section width as effective flow area. Original HEC-2 model construction (from which the UNET model was derived) was based on an effective flow width within the leveed area. Cross section width generally varies from 10,000 to 20,000 feet. However, due to the numerous natural and constructed obstructions within the conveyance area, effective flow width is much less than the cross section width. As a result, the no levee model overstates the available flow area. Reach lengths between sections are also incorrect as the flow conveyance area is dramatically shifted. Computed results for the levee removal alternative should be regarded as rough estimates only. Correct simulation of this alternative would require the construction of an entirely new model and detailed studies to determine the long term effects of vegetation and sedimentation within the floodplain on conveyance.

11.1.3. Levee Removal Implementation. Cost analysis performed for removal of the existing levee assumed that 10% of the existing levee would be removed to provide sufficient conveyance beyond the existing levee alignment. A figure of 10% corresponds to removal of approximately a 200 foot levee segment within every 2000 feet. Actual levee removal areas would be site specific dependent on channel and levee alignment.

11.2. Levee Confinement.

For this alternative, all agricultural levees were raised infinitely high such that the 1993 flood event was confined to the existing area between the levees. In the Rulo, NE area, where no federal levees are present, the private levees were raised. Levee locations or roughness values were not altered for this alternative. An additional 3 feet was added to the confined elevation for the construction levee height when performing cost analysis.

11.3. Levee Height at 25 Year Level.

For this alternative, the height of all agricultural levees were set to correspond with an estimated 25 year profile for the Missouri River. The 25 year profile was developed from profiles contained in the Special Flood Hazard Information, Missouri River, Gavins Point Dam to Rulo, Nebraska, Volumes I and II. Federal levees, which are currently higher than the 25 year elevation, were notched to an elevation equal to the 25 year elevation. Levees lower than the 25 year within the Rulo, NE area left bank were raised to the 25 year elevation. The levee notch was designed as an erodible plug. When flood levels exceed the 25 year level, the levee notch is eroded and the cell fills with water. In this manner, the levee cells along the channel act as detention basins to store flows which exceed the 25 year event. For the 25 year notch alternative, the computed 25 year elevation should be used for the levee height.

11.3.1. Implementation Assumptions. For implementation, each levee cell was assumed to include a constructed notch at the upstream and downstream end. The notch would consist of a lowered section which would act as a fuse plug of erodible material. The notch would consist of an erodible core material overlaid with a top layer. The notch would be designed to fail in a non-catastrophic manner. For cost purposes, the notch construction width was assumed to consist of an average 500 foot width and extend to an elevation 5 feet below the ground elevation at the toe of the levee. The downstream notch would be constructed at the 25 year elevation. The upstream notch would be constructed at the 25 year elevation plus 3 feet. Levees which must be raised (are currently below the 25 year level) should be constructed at the 25 year elevation without any freeboard. Table OM-8 shows the federal levee designation, river mile location, and elevation of the 25 year notch employed within the UNET model.

11.3.2. UNET Implementation. Within the UNET model, the 25 year notch alternative was modeled by creating simple embankment failure records (SF record) for each notch location. When the computed water surface at the notch location equals the elevation specified on the SF record, failure is initiated. For modeling purposes, all failures assumed that the erodible plug would function such that the interior levee cell would fill in a 24 hour time period.

12. Upland Retention/Watershed Measures.

Various policy and structural measures exist which may affect inflow rates to the river system. For the evaluation of these measures, no modifications to UNET model geometry were performed. Assessment was performed by adjusting inflow hydrographs to the UNET model for each scenario

examined.

12.1. Revise 1993 Reservoir Releases.

During the July 1993 peak flooding period, reservoir releases from Gavins Point Dam averaged 8000 cfs. Release volume from Gavins Point Dam totaled 2.06 million acre-feet from June through August, 1993. Reservoir release rates corresponded with minimal releases required for downstream water uses. The minimal flow released from Gavins Point Dam had no effect on downstream flood levels. Further reduction of reservoir releases during the 1993 flood event would not have been practical or beneficial. Refer to the 1993 - 1994 Missouri River Mainstem Reservoir Annual Operating Plan report for details regarding system inflow, pool levels, and operation of the mainstem reservoirs.

Table OM-8
25-Year Levee Height (FT @ M.S.L.)
Missouri River from Omaha to Rulo, NE

Missouri River Federal Levee	Storage Area Invert	Upstream End		Downstream End	
		River Mile	Levee Elevation	River Mile	Levee Elevation
L-611-614	950.0	594.0	965.8	588.9	956.7
L-601	945.0	587.6	958.4	580.1	948.9
L-594	939.0	580.0	951.2	573.9	940.4
L-575 UP	920.0	572.9	942.6	562.0	928.3
L-575 DN	900.0	561.0	929.5	544.0	909.4
R-573	910.0	556.5	923.8	552.0	917.5
R-562	903.0	548.9	917.3	543.0	908.5
L-550 UP	890.0	542.1	910.7	535.3	901.7
L-550 DN	882.0	535.1	903.0	522.2	887.0
R-548	887.0	534.5	902.2	528.3	894.3
L-536	874.0	521.9	889.6	516.3	880.1
R-520	863.0	505.4	872.2	501.7	865.5

Note: The tabulated levee river miles and stage are for the 25-year UNET model alternative and are not actual levee parameters.

12.2. Without Federal Reservoirs.

Analysis was performed to assess the affect of reservoir storage on peak flow rates during the 1993 event for the base and wet antecedent conditions. Within the Omaha District, major Federal reservoirs include the six mainstem dams on the Missouri River upstream of Gavins Point Dam at river mile 811.1. The Missouri River Division Reservoir Control Center (RCC) annually computes the without reservoir hydrograph at Gavins Point Dam based on routed upstream inflows. UNET modeling was performed employing the without reservoir flow hydrograph computed by RCC for inflow into the model instead of the actual 1993 reservoir releases. All other parameters were unchanged from the base condition. The without reservoir hydrograph computed by RCC at Gavins Point Dam did not contain any

large peaks flows during the 1993 event. Discharge generally varied from 60,000 - 90,000 cfs for a 3 month period. Essentially, the without reservoir hydrograph is equivalent to adding substantial base flow to the Missouri River for the 1993 event.

12.3. Reservoir Antecedent Conditions.

A brief analysis was performed to evaluate 1993 reservoir releases from the six mainstem dams for different antecedent conditions in the upper Missouri River basin. An extended drought occurred in the upper Missouri River basin from 1987 through 1992. At the beginning of March 1993, reservoir storage within the six mainstem reservoirs was at 43.0 million acre-feet (MAF) or 12.4 MAF below normal. The lowest reservoir storage total since 1967 when all the reservoirs were first filled to their normal operating pool was 40.8 MAF which occurred in January, 1991. During the 1993 flooding on the Missouri and Mississippi Rivers, the Missouri River mainstem reservoir system stored a significant volume of runoff. Gavins Point Dam released minimal flows well below normal releases for the flood period in order to alleviate downstream flooding to the maximum extent possible. During the 1993 flood, 2.2 million ac-ft was released from Gavins Point Dam. These releases are considered the minimal possible and would be different for other antecedent conditions.

An analysis was performed to evaluate reservoir releases for the following antecedent conditions in the upper Missouri River basin: 1) reservoir pools at or near normal levels at the start of the 1993 flood and 2) if conditions had been such that the reservoir pools were at the base of exclusive flood control pool elevations. This simplified analysis did not take into account all the various factors involved in operating the mainstem reservoirs such as required reservoir releases, distribution and timing of the runoff, and the various operating constraints of the reservoir system. Operational criteria for the mainstem system is outlined in the Missouri River Mainstem Master Manual. Results of the evaluation are summarized in table OM-9.

12.3.1. Normal Conditions. Normal antecedent conditions was assumed to be represented by reservoir pool levels at the historical average end of month pool elevation for May (based on 27 years of record) instead of the lower 1993 levels which were due to drought conditions. At normal May end of month pool levels, there is approximately 14.4 million acre-feet of available storage in the six reservoirs. This would have been sufficient capacity to hold almost all of the 12.5 million acre-feet inflow into the reservoirs during the period June through August, 1993. At the lowest reservoir, Gavins Point Dam, excess inflow from the Niobrara River would have been within what was released during the 1993 operation of the reservoirs. Although operation procedures may have varied, analysis determined that the excess inflow into Gavins Point Dam would have been less than the volume released during the actual 1993 operation of the reservoirs.

Assuming normal pool levels and following the current reservoir regulation criteria, a simple analysis determined an addition release of 10,000 to 20,000 cfs. The additional release is very minor and is only 2 to 4 percent when compared to the Missouri River peak flow of 300,000 cfs at Rulo, NE (RM 498), and 750,000 cfs peak discharge at Hermann, MO, (RM 97.9). Therefore, a significant increase in releases in 1993 which would impact downstream flood levels would not have been required if initial pool levels had been at normal levels.

Table OM-9 Available Storage Based on the Average End of Month Pool for May Missouri River Mainstem Reservoirs [1]					
Mainstem Reservoir	**Average EOM Pool for May (Ft M.S.L.)**	**May 31 1993 Pool Elevation (Ft M.S.L.)**	**Exclusive Flood Control Pool Top Elev. (Ft M.S.L.)**	**Total Storage Volume Available[2] (Ac-Ft)**	**Total Reach Inflow Vol.[3] Jun-Aug 1993 (Ac-Ft)**
Fort Peck	2234.9	2213.3	2250.0	3,141,000	3,460,000
Garrison	1836.7	1822.9	1854.0	5,595,000	5,920,000
Oahe	1607.2	1600.2	1620.0	3,874,000	1,900,000
Big Bend	1420.5	1420.9	1423.0	117,000	110,000
Fort Randall	1357.2	1355.7	1375.0	1,579,000	430,000
Gavins Point	1205.6	1206.1	1210.0	110,000	680,000

Note: 1 Reservoir data based on available information and is subject to change.
2 Refers to the available storage volume between the May average end of month pool elevation and the top of the exclusive flood control elevation at each of the reservoirs.
3 Reservoir inflow volume in excess of reservoir storage is a controlled release according to reservoir regulation criteria as experienced during normal operation.

12.3.2. Wet Conditions. Extremely wet antecedent conditions were assumed to be represented by reservoir pool levels at the exclusive flood control pool elevation. Assuming all six reservoir levels at the exclusive flood control pool level would constitute an extremely rare event that, based on operation of the reservoirs in compliance with the Missouri River Main Stem Master Manual, would be highly unlikely. In the 27 years since all the reservoirs were filled to their normal operating pool, the end of month May pool elevation at everyone of the six mainstem reservoirs has been below the elevation of the exclusive flood control pool.

If antecedent conditions had been such that only the exclusive flood control zone were available in the mainstem Missouri River Reservoirs and ignoring the timing of inflow with releases, a simple volume analysis determined that approximately one third of the inflow would have been captured by the reservoirs. Actual 1993 operation captured approximately 80% of the inflow. Although capacity to store near 100% of the inflow was available in 1993, minimal releases during the summer months were necessary for downstream water uses. The no reservoir alternative modeled with UNET assumed 0% capture of inflow. Reservoir releases for extremely wet conditions is bracketed between computed results for the base and the no reservoirs alternative UNET models.

12.3.3. Conclusion.

Analysis was conducted to evaluate the effect of reservoir releases for different antecedent conditions in 1993. A simple volume analysis determined that additional releases of a magnitude which

would have significantly impacted downstream flood levels would not have been required if pool levels had been at normal levels. Although reservoir regulation criteria may have required additional reservoir releases, the release rate would have been minimal in comparison to the magnitude of the 1993 event. Extremely wet antecedent conditions was represented by pool levels at the exclusive flood control zone. In this case, approximately 1/3 of the total inflow to the reservoir system would have been stored. However, reservoir pool levels within the exclusive flood control zone at the end of May has not occurred at anyone of the six dams and should be regarded as an extremely rare event. Downstream impacts would be bracketed between the UNET model computed results for the base condition and the no reservoirs alternative. The examination of antecedent conditions illustrates that, with the exception of extremely rare circumstances, mainstem Missouri River reservoir volume would usually allow a release schedule of minimal releases which would not have significantly impacted downstream flood levels during the 1993 event.

12.4. Runoff Reduction.

For this alternative, measures which would reduce the total runoff volume during the 1993 flood were evaluated by reducing mainstem and tributary inflow hydrographs to the model by both 5 and 10 percent. Based on the St. Paul District's preliminary studies of wetland storage and other upland retention methods, it was determined that the maximum reasonable amount of available storage would reduce the total runoff volume into the Mississippi and Missouri Rivers of between 5 and 10 percent. Depending on individual drainage basin characteristics, some tributary basins could store more than 10 percent of the basin runoff volume and some tributary basins have little or no upland retention storage available. To simplify the UNET modeling, all the inflow hydrographs were reduced by an equal amount. Also, in reality, runoff reduction would not be distributed equally over the total inflow hydrograph but instead would have a major impact on the shape of the inflow hydrograph at the beginning of the 1993 event and would have little or no impact on the peak discharges and stages on the river. No tributaries in the Missouri River basin of the Omaha District were studied to determine the amount of available upland retention storage.

13. Levee Setbacks.

The UNET model was employed to analyze the effect on flow conditions throughout the study reach for both an isolated levee setback and a systemic setback of all levees on the Mississippi and Missouri Rivers. Within the Omaha District, levees were setback from the Missouri River channel bank a distance of 1000 - 3000 feet when originally constructed. Setback of a levee refers to moving the levee from the present location to a new location which is further from the river. Levee setbacks are intended to increase the cross section flow width instead of constricting the flow area to a narrow channel. However, the flow area increase may be offset by an elevated roughness condition. The levee setback analysis did not address any changes in flow roughness. The case study of an isolated levee setback analysis illustrated how undesirable consequences may occur on the entire system from modifying a small section of the channel. The systemic levee setback illustrates the effects on the total system by setting all levees back. The isolated levee setback analysis was performed to determine the impacts of an isolated levee setback on computed peak flows and stages at downstream locations. The systemic levee setback analyzed setback impacts on a system wide basis. The two analyses are discussed in the following paragraphs.

13.1. Case Study - Isolated Levee Setback

13.1.1. Isolated Levee Setback Reach. The location selected for the case study was the L-550 and L-536 federal levee units. The levee units are located on the left bank of the Missouri River between RM 542.1 and RM 516.3. Within the reach, Missouri channel width averages 800 feet. Federal levees are setback from the channel between 1000 and 3000 feet. Private levees have been constructed adjacent to the Missouri River channel bank within most of the reach. The area between the private levee and the federal levee is generally agricultural row crops. In the 1993 event, L-550 levee capacity was exceeded for a significant period of time with levee overtopping for a total length of 1-2 miles at a depth of 1-2 feet. In the morning of 24 July, the L-550 levee breached approximately 1.5 miles upstream of Brownville, NE at RM 536.7. Levee unit L-536 did not overtop or breach during the 1993 event. Private levees within the Rulo overbank area downstream of L-536 suffered extensive damage and essentially had no constricting effect on flow during the peak flow period.

13.1.2. Levee Setback Parameters. Levee setback distance was determined by computing how much the water surface elevation was lowered as a result of the levee setback. The levee breach at L-550 was assumed to have been directly dependent upon water surface level. Therefore, ignoring the effects of duration and seepage, the levee was assumed to remain intact if computed water surface elevation was less than the elevation at which overtopping occurred in 1993. Within the UNET model, levee failure corresponded to a water surface elevation of 903.1 at RM 535.25. Brief iterative analysis indicated that a levee setback distance of 3000 feet lowered the water surface in the setback reach that overtopping and failure did not occur.

Levee setback was performed from the upstream end of unit L-550 at RM 542.1 to the downstream end of unit L-536 at RM 516.3. Federal levee unit L-536 is the furthest downstream levee on the left bank within the Omaha District. Downstream of unit L-536, the Rulo overbank area contains only private levees. The downstream end of the levee setback was selected to provide a reasonable tie-in point and minimize downstream impacts. For cost analysis, the existing levee was removed at the 10% ratio employed within the levee removal alternative. Construction of a new levee was assumed along the setback alignment.

13.1.3. UNET Implementation. Levee setback was modeled by changing the encroachment station on the X3 record within the UNET model. The existing levee was not physically removed from the cross section. The entire setback distance of 3000 feet was assumed to be effective flow area within each cross section. Accounting for flow width reductions due to levee alignment changes and other constraints was not performed.

Increasing the setback distance by 3000 feet would affect roughness values within the cross section. Estimating what cross sectional changes would occur such as vegetative growth, sediment deposition, and etc. is highly speculative and was not investigated. No changes were made to model calibrated discharge-conveyance relationships within the levee setback area. Roughness for the area between the existing levee and setback levee locations may increase due to changes in land use which would increase stages in the area of the setback. Unless the Missouri River bank private levees were removed, the area between the federal levee and the riverbank would probably remain agricultural row crop. The possible combinations of land use, geometry, and roughness changes were not examined for their effect on computed results. All changes in roughness and geometry which may be caused or aggravated by the levee setback were ignored.

13.2. Systemic Levee Setback.

13.2.1. Levee Setback Reach. The location for the systemic levee setback included all agricultural levees on both the Mississippi and Missouri Rivers within the FPMA study limits. For the Omaha District, this included setting back the levees from south of the urban levees at Omaha, NE and Council Bluffs, IA (RM 600.0) to the downstream boundary of the district near Rulo, NE. (RM 498.0). The systemic levee setback continued downstream of Rulo, NE through the Kansas City district to the confluences of the Missouri and Mississippi Rivers. For the existing federal levee system within the Omaha District, distances between the levees ranged from 1,050 feet to over 13,900 feet, with an overall average distance between the federal levees of about 5,600 feet. For the Rulo overbank private levees, which are adjacent to the Missouri River channel bank, distances between levees ranged from just over 700 feet to over 7,000 feet with an average about 1,900 feet.

13.2.2. Levee Setback Parameters. To determine the minimum 5,000 feet levee setback, the levees on the left and right banks of the Missouri River were measured for each cross section to determine the existing distance between the levees. Distances between levees that was less than 5,000 feet were set to a minimum distance of 5,000 feet. For cross sections with a distance between the levees of greater than 5,000 feet, the levee location was not changed.

13.2.3.UNET Implementation. Many of the modeling assumptions used for the case study levee setback as discussed in Section 13.1.3 also were used for the systemic levee setback. As was done with the isolated levee setback, the cross section was not physically changed and only the left bank encroachment stationing of the X3 record was changed to meet the 5,000 feet minimum requirement. The UNET model was setup for two modeling alternatives. The first alternative consisted of using the top of levee elevations of the existing levees for the setback levees. For the second alternative, the setback levee top elevations were raised to fully contain the 1993 flood event between the levees without any levee breaching. Again, all changes in roughness and geometry which may be caused or aggravated by the levee setback were ignored by the analysis.

14. Interior Drainage.

Flooding to the Missouri River overbank is quite apparent when a levee is overtopped or fails. However, even if a levee does not overtop or fail, the interior area behind the levee may still experience extensive flooding when high stages on the Missouri River prevent drainage structures through the levee from removing interior runoff. An example of this would be Hamburg, IA where flooding by the Missouri River and Nishnabotna River was prevented by Federal levee L-575 and its tieback. However, the lack of ability to drain the interior area due to high stages on the Nishnabotna and Missouri Rivers blocking the drainage structures through L-575 and rainfall amounts of over 18 inches during July 1993, caused extensive flooding to the city of Hamburg and surrounding agricultural lands. This was also the case behind many of the federal levees on the Missouri River.

For the various alternatives, the altering of the stage hydrograph on the Missouri River will affect the interior areas behind the federal levees by changing ponding depths and duration. Two of the major effects are Missouri River backwater effects on the outflow of levee drainage structures and changing the levee seepage rate caused by Missouri River stages. Also, the elimination of the interior runoff through the use of pumps was examined.

14.1. Drainage Structures.

The operation of the existing drainage structures may be altered by the different alternatives. A

case study of a typical interior drainage structure through the federal levee was performed to illustrate impacts of the various alternatives on interior drainage. The invert of a drainage structure through levee L-575 at river mile 554.4 was compared with the Missouri River stage hydrographs for existing conditions, 10 percent runoff reduction and no reservoirs alternatives. This would bracket the greatest potential change of the stage hydrograph.

As can be seen from the graph on plate OM-18, the stage of the Missouri River is so great and the duration so long that altering the stage hydrograph would not have helped or hampered the functionality of the existing drainage structures during the 1993 flood. The duration of flow above the invert of the drainage structure at RM 554.4 was for at least the months of June through August, 1993 for existing conditions, and for both, the 10 percent reduction alternative and no reservoirs alternative. Because the interior water at this location ponded to about elevation 910 feet @ m.s.l., there would have been one additional day that water could have drained for the 10 percent reduction alternative. Since the baseline condition was below elevation 910 for about 25 days during August, this would represent an increase of about 4 percent in the duration which the outlet could have drained during the 1993 event if the inflow were reduced by 10 percent.

14.2. Seepage.

When the Missouri River is high over an extended period of time, seepage of water into the levee-protected lands becomes a problem. Because seepage occurs when gravity drainage is not possible, pumping or ponding are the only alternatives for addressing the problem. The three important factors for seepage are Missouri River stages, duration of high stages, and seepage rates.

For the alternatives, the altering of the flood hydrograph would alter the amount of seepage that may occur depending upon the change in stage and the duration thereby increasing the amount of seepage into an interior area. As an example, the stage increase at Rulo, NE for the confined levees alternative was compared to the existing conditions to determine the maximum increase in stage and duration. Using data obtained from the seepage analysis for the Thurman to Hamburg Study (USACE,1993) for Federal levee L-575 the two to seven feet increase in stage for the 20 extra days would add approximately 1,200 acre-feet of seepage into the Rulo overbank area. This assumes a levee length of about 35 miles (RM 515 to RM 480). With an area of about 88,600 acres and runoff for the month of July, 1993 being over one foot, this would add less than two percent to the total volume of water in the Rulo overbank. Therefore, the negative impact of seepage into the overbank areas caused by increasing the stage on the Missouri River is considered negligible.

14.3. Pumping.

As discussed earlier, when the Missouri River is high over an extended period of time, many of the drainage structures through the federal levee cannot drain properly. The two major alternatives for removing interior runoff when drainage structures are not functioning are pumping and/or ponding. Due to the limited extent of this study, ponding or the combination of pumping and ponding was not investigated.

One of the alternatives is confined levees with no overtopping or failure of the levees. This will not allow any water from the Missouri River to flood the overbank behind the levee. However, interior flooding from excess runoff could be significant as shown in Hamburg. As an example of the amount of pumping capacity required to remove the interior drainage runoff from behind the federal levees for

the July 1993 rainfall event, data obtained from the Thurman to Hamburg, for Main Ditch 6 was used. Main Ditch No. 6 is a 67 square mile basin that drains through levee L-575. An HEC-1 model was setup to simulate various pumping sizes required to remove the daily interior runoff volumes from the July 1993 rainfall. The criteria for pumping is to not allow any agriculture land to be inundated for more than 48 hours. On average, crops that are under water for longer than 48 hours are considered destroyed. Based on this criteria, to fully remove the interior runoff from the 18 inches of rain (minus infiltration) that fell on the Main Ditch 6 basin during July, 1993, pumps with a total capacity of approximately 4000 cfs would be required. While this is not practical from the standpoint the Main Ditch 6 capacity is about 1,000 cfs, it does give an idea of the magnitude of the 1993 event and how very little could have been done to relieve interior flooding.

To apply this to other interior areas on the Missouri River overbank, the 4000 cfs pumping capacity was divided by the 67 square mile basin area. This would require a pumping capacity of about 60 cfs per square mile of drainage area. The total overbank area behind federal levees and private levees between Omaha, NE and Rulo, NE is 414 square miles. The total overbank area was multiplied by an additional 20 percent to account for the runoff from the hills. Therefore, the pumping requirements for the total area of approximately 500 square miles would be about 30,000 cfs.

It should be noted that this is a very rough estimate based on one location on the Missouri River overbank being applied to the whole basin. Alternatives to pumping, such as ponding areas, were not evaluated.

15. Summary of Results.

Output of the UNET model consists of hydrographs at specified locations, maximum flow and water surface elevation profiles for each reach, storage cell stage hydrographs, and levee connection flow hydrographs. Computed data from the UNET model was extracted and summarized to allow the evaluation of the base and alternative conditions. Plates OM-19 through OM-21 illustrate a graphical representation of Missouri River and cell peak stage variation from the base condition at selected locations.

15.1. Hydrographs.

Plotted stage hydrographs at selected locations for the base condition and various alternatives are shown on Plates OM-22 to OM-44.

15.2. Peak Flow and Stage.

A tabulation of Missouri River peak flows and stages for the base condition and various alternatives are shown on tables OM-10 and OM-11. Tabulation location corresponds with the gaging station locations. In addition, computed results at river mile 520 were also tabulated to illustrate the location upstream of where the federal levees end near river mile 515. When examining results in the Rulo, NE area, it was noted that the low levees in the area were overtopped early in the storm event and had very little effect on conveyance during the peak of the flood event. Therefore, the Rulo overbank reach acted as a natural conveyance reach for much of the 1993 event.

15.3. Levee Cell Stage.

A tabulation of the peak levee cell stages for the various alternatives are shown on Table OM-12. Levee cell stages are computed by the UNET model based on cell inflow\outflow and cell stage/storage. Computed peak levee cell stages are not identical with computed Missouri River peak stages.

Table OM-10
Maximum Discharge (CFS)
Missouri River from Gavins Point to St. Joseph, MO

Missouri River Location	River Mile	Computed	Observed	Levees Removed		Confined	25 Year	No Reservoirs	Runoff Reduction		Case Study Setback	Systemic Levee Setback	
				Agriculture	Natural				5%	10%		Existing	Confined
Yankton, SD	805.8	25,300	26,600 5.1%	--------	--------	--------	--------	97,400 285.0%	--------	--------	--------	--------	--------
Sioux City, IA	732.3	72,000	72,200 0.2%	--------	--------	--------	--------	145,000 101.0%	--------	--------	--------	--------	--------
Decatur, NE	691.0	76,600	76,400 -0.3%	--------	--------	--------	--------	147,700 92.8%	--------	--------	--------	--------	--------
Omaha, NE	615.9	112,900	115,000 1.9%	112,900 0.0%	112,900 0.0%	112,900 0.0%	112,900 0.0%	177,000 47.9%	107,300 -5.0%	101,600 -10.0%	113,100 0.2%	112,900 0.0	112,900 0.0
Plattsmouth, NE	591.5	189,600	--------	187,400 -1.2%	182,200 -3.9%	189,600 0.0%	177,300 -6.5%	251,600 32.7%	180,200 -5.0%	170,600 -10.0%	191,200 0.8%	190,400 0.4%	190,400 0.4%
Nebraska City, NE	562.6	202,700	196,000 -3.3%	192,900 -4.8%	180,700 -10.9%	203,000 0.1%	200,900 -0.8%	279,300 37.8%	191,500 -5.5%	179,500 -11.4%	200,500 -1.1%	204,900 1.1%	204,900 1.1%
Brownville, NE	535.3	236,100	--------	234,100 -0.8%	218,300 -7.5%	245,000 3.8%	198,400 -16.0%	249,000 5.4%	217,100 -8.0%	211,700 -10.3%	244,000 3.3%	225,700 -4.4%	245,200 3.9%
River Mile 520	520.0	302,800	--------	290,800 -4.0%	259,200 -14.4%	308,200 1.8%	239,300 -21.0%	286,100 -5.5%	271,800 -10.2%	273,300 -9.7%	307,200 1.5%	283,200 -6.5%	308,500 1.9%
Rulo, NE	498.1	167,700*	307,000 na	291,100 na	258,600 na	319,000 na	145,600* -13.2%	164,400* -2.0%	152,900* -8.8%	153,200* -8.6%	234,600* 40.0%	228,500* 36.3%	318,600 na

Note: Top value is maximum discharge. Bottom value is the percent change from the computed discharge.

Note: * - Significant discharge bypassed the Missouri River channel at Rulo, Ne and went through the Rulo overbank area.

Note: Case Study Setback refers to an isolated setback of federal levees L-550 and L-536. Systemic Levee Setback refers to a minimum 5,000 foot setback of all federal and private levees on the Missouri River through out the Omaha District. For the Systemic Levee Setback Alternatives, Existing refers to existing levee heights. Confined refers to a levee height tall enough to confine the 1993 flood event without any levee failures.

Table OM-11
Maximum Stage (Feet @ M.S.L.)
Missouri River from Gavins Point to St. Joseph, MO

Missouri River Location	River Mile	Computed	Observed	Levees Removed		Confined	25 Year	No Reservoirs	Runoff Reduction		Case Study Setback	Systemic Levee Setback	
				Agriculture	Natural				5%	10%		Existing	Confined
Yankton, SD	805.8	1157.2	1154.8 -2.4	-------	-------	-------	-------	1162.4 5.2	-------	-------	-------	-------	-------
Sioux City, IA	732.3	1084.5	1084.3 -0.2	-------	-------	-------	-------	1092.8 8.3	-------	-------	-------	-------	-------
Decatur, NE	691.0	1042.3	1042.2 -0.1	-------	-------	-------	-------	1049.5 7.2	-------	-------	-------	-------	-------
Omaha, NE	615.9	978.5	978.5 0.0	978.2 -0.3	978.6 0.1	978.5 0.0	978.5 0.0	983.4 4.9	977.8 -0.7	977.1 -1.4	978.5 0.0	978.5 0.0	978.5 0.0
Plattsmouth, NE	591.5	964.3	964.5 0.2	961.2 -3.1	963.6 -0.7	964.3 0.0	963.3 -1.0	967.7 3.4	963.7 -0.6	962.9 -1.4	963.7 -0.6	963.9 -0.4	963.9 -0.4
Nebraska City, NE	562.6	932.6	932.6 0.0	927.9 -4.7	930.3 -2.3	932.6 0.0	930.8 -1.8	935.1 2.5	931.9 -0.7	931.1 -1.5	930.7 -1.9	931.2 -1.4	931.2 -1.4
Brownville, NE	535.3	904.2	904.3 0.1	900.5 -3.7	903.4 -0.8	904.8 0.6	901.7 -2.5	904.2 0.0	902.9 -1.3	902.9 -1.3	902.3 -1.9	903.2 -1.0	904.4 0.2
River Mile 520	520.0	889.2	-------	885.6 -3.6	887.6 -1.6	890.8 1.6	886.1 -3.1	888.5 -0.7	887.9 -1.3	888.0 -1.2	888.3 -0.9	888.1 -1.1	889.4 0.2
Rulo, NE	498.1	862.8	862.6 -0.2	861.5 -1.3	864.8 2.0	870.0 7.2	860.2 -2.6	862.7 -0.1	862.3 -0.5	862.3 -0.5	863.8 1.0	863.8 1.0	867.1 4.3

Note: Top value is maximum stage. Bottom value is the change in stage from the computed stage.

Note: Case Study Setback refers to an isolated setback of federal levees L-550 and L-536. Systemic Levee Setback refers to a minimum 5,000 foot setback of all federal and private levees on the Missouri River through out the Omaha District. For the Systemic Levee Setback Alternatives, Existing refers to existing levee heights. Confined refers to a levee height tall enough to confine the 1993 flood event without any levee failures.

Table OM-12
Maximum Stage Behind Federal Levees (FT @ M.S.L.)
Missouri River from Omaha to Rulo, Nebraska

Missouri River Federal Levee	River Mile	Area Protected (Acres)	Storage Area Invert	Computed	25 Year	No Reservoirs	Runoff Reduction		Case Study Setback	System Levee Setback
							5%	10%		
L-611-614	588.0 - 596.0	10,920	950.0	950.0 0.0	960.9 10.9	950.0 0.0	950.0 0.0	950.0 0.0	950.0 0.0	950.0 0.0
L-601	588.0 - 580.1	13,800	945.0	945.0 0.0	945.0 0.0	945.0 0.0	945.0 0.0	945.0 0.0	945.0 0.0	945.0 0.0
L-594	573.5 - 580.1	9,900	939.0	939.0 0.0	940.6 1.6	947.6 8.6	939.0 0.0	939.0 0.0	939.0 0.0	939.0 0.0
L-575 UP	561.8 - 573.5	36,680	920.0	920.0 0.0	925.1 5.1	926.1 6.1	920.0 0.0	920.0 0.0	920.0 0.0	920.0 0.0
L-575 DN	543.8 - 561.8	35,400	900.0	900.0 0.0	912.1 12.1	916.1 16.1	900.0 0.0	900.0 0.0	900.0 0.0	900.0 0.0
R-573	553.0 - 557.0	2,200	910.0	910.0 0.0	916.7 6.7	910.0 0.0	910.0 0.0	910.0 0.0	910.0 0.0	910.0 0.0
R-562	542.0 - 549.0	6,800	903.0	903.0 0.0	910.3 7.3	913.3 10.3	903.0 0.0	903.0 0.0	903.0 0.0	903.0 0.0
L-550 UP	535.2 - 542.0	20,500	890.0	902.1 12.1	900.9 10.9	902.0 12.0	901.1 11.1	900.7 10.7	890.0 0.0	901.5 11.5
L-550 DN	522.0 - 535.2	17,800	882.0	892.2 10.2	889.0 7.0	891.6 9.6	888.1 6.1	886.1 4.1	882.0 0.0	889.5 7.5
R-548	528.0 - 534.0	2,900	887.0	887.0 0.0	895.1 8.1	900.3 13.3	887.0 0.0	887.0 0.0	887.0 0.0	887.0 0.0
L-536	516.0 - 522.0	12,520	874.0	874.0 0.0	879.8 5.8	874.0 0.0	874.0 0.0	874.0 0.0	874.0 0.0	874.0 0.0
R-520	501.0 - 504.0	1,600	863.0	863.0 0.0	863.0 0.0	863.0 0.0	863.0 0.0	863.0 0.0	863.0 0.0	863.0 0.0

Note: Top value is maximum stage. Bottom value is the change in stage from the storage area invert

Note: Case Study Setback refers to an isolated setback of federal levees L-550 and L-536. Systemic Levee Setback refers to a minimum 5,000 foot setback at existing levee elevation of all federal and private levees on the Missouri River throughout the Omaha District.

15.4. Flood Boundaries.

An approximate outline of flood boundaries were developed employing CADD technology for the Omaha, NE to Rulo, NE reach. Topographic representation of the study area was obtained from scanned USGS 7.5 minute quadrangle topographic maps. Quad map contour interval varied from 5 to 10 foot. Intergraph software was employed to develop a digital terrain model from the scanned quadrangle maps. For areas behind the federal levee cells, flood boundaries were determined utilizing the peak stage determined within the levee cell by the UNET model. Interior drainage was not modeled by UNET and not considered when determining flood boundaries. For the area near Rulo, NE, flood boundaries were determined employing the sloping water surface profile computed by the UNET model for the Missouri River. Flood boundaries for the levee removal alternatives were also developed employing the sloping Missouri River water surface profile. The density of available topography from the quadrangle sheets restricted the accuracy of the flood boundaries. Employing CADD software greatly reduced the time required to develop flood boundaries for the 120 miles from Omaha, NE to Rulo, NE. Software and hardware limitations encountered when utilizing digital topographic representation for such an extremely large area required the use of relatively coarse feature representation to reduce the vast amount of data to acceptable levels. As a result, flood boundary accuracy was further restricted. A depiction of computed flood boundaries for the base condition and each alternative are displayed on plates OM-45 to OM-52.

16. Discussion of Results.

Examination of the results illustrated several interesting aspects of applying the alternatives on a system wide basis. Results were examined to compare base and alternative conditions with respect to hydrograph timing, peak flow, peak stage, levee overtopping and stages within levee cells, and flood duration. When comparing alternatives, all parameters such as peak flow, stage, and levee cell stage must be examined throughout the entire reach to completely evaluate the effects of each alternative. Specific comments for each alternative are as follows:

16.1. Levee Removal.

Levee removal provides a means of reducing computed stages. Stage reduction is extremely dependent upon floodplain use as results from the low and high loss conditions illustrates. Discharge variation caused by levee removal is also dependent upon floodplain roughness.

16.1.1. Stage. Levee removal generally reduced computed stages. Between Plattsmouth and the downstream federal levee end (RM 520), stage reduction varied from 3 - 4.5 feet for the low roughness and from 0.7 to 2.3 feet for the high roughness. The largest stage reduction occurred at Nebraska City, NE. The Rulo, NE area contains some structures, road embankments, railroads, and other obstructions within the floodplain and represents a lower roughness area with few expansion/contraction losses. At Rulo, NE, the existing private levees were non-effective and severely damaged in 1993 to the extent that the floodplain was essentially a "no levee" condition. Comparing model results to the actual 1993 stages, the model stage was 1.3 feet <u>lower</u> for the agricultural condition and 2.0 feet <u>higher</u> for the natural condition. Results at Rulo, NE illustrate that the high and low loss values bracket the actual stage and provide a basis for expected results in a "no levee" condition.

16.1.2. Discharge. For the low loss condition, peak discharge was generally very close to the base condition and only varied by approximately 10,000 cfs. The reduction in flow varied from -1.2% at Plattsmouth, NE to -4.8% at Nebraska City, NE. For the maximum loss condition, the peak

discharge was reduced by 20,000 cfs between Nebraska City, NE and Brownville, NE and by 40,000 cfs at Rulo, NE. The reduction in flow varied from -3.9% at Plattsmouth, NE to -14.4% near Rulo, NE. The increased flow reduction corresponds with increased stages.

16.1.3. Hydrograph Timing. For the levee removal condition, the occurrence of peak stages were shifted later by 24 - 72 hours. Peak stages occurred at earlier for the low loss condition than the high loss conditions. As a result of the hydrograph timing shift, different inflow conditions would produce different results. Potentially, peak stages could actually increase for the levee removal condition if peak coincidental timing of Missouri River and inflow hydrographs occurred.

16.2. Levee Confinement.

Within the majority of the Omaha District, flows were confined between the levees for the 1993 event. Within these areas, no changes were computed for the total confinement alternative. The exception is the L-550 levee, which failed, and the Rulo, NE area, where no federal levees were constructed.

16.2.1. Stage. No changes were observed from the base condition except in the reaches downstream of Brownville, NE. These results are consistent with the fact that no federal levees overtopped or failed in the reach from Omaha, NE to just upstream of Brownville, NE. In the federal levee area downstream of Brownville, NE, stages increases were minor and averaged near 1 foot. In the Rulo, NE area, where the flow width varied from 3-7 miles for the 1993 event, confining the flow to a narrow leveed width caused a large stage increase of nearly 8 feet.

16.2.2. Discharge. Peak discharge for most locations were similar between the base and confined conditions. At Brownville, NE, there was a 3.8% increase in flow due to no levee failure on L-550. Near Rulo, NE, there was about a 2.0% increase in discharge computed.

16.2.3. Hydrograph Timing. The levee confined condition did not appear to significantly alter the hydrograph timing from the 1993 base condition.

16.3. Levee Height at 25 Year Level.

Reduction in peak stage and discharge was computed. These reductions were possible as a result of the failure of 10 additional levee cells and the inundation of a significant area between Omaha, NE and Rulo, NE. At St. Joseph, MO, the furthest downstream point employed for comparing results, stage and discharge reductions for this alternative exceeded reductions computed for the levee removal alternative.

16.3.1. Peak Stage. Setting levee elevations at the 25 year levee resulted in a stage reduction for the Missouri River. Within the federal levee reach, stage reduction increased in the downstream direction from a value of 1.0 feet at Plattsmouth to 3.1 feet near the downstream federal levee end (RM 520). Results illustrate that lowering the federal levees to a 25 year level and detaining flow within the overbank levee cells has the potential for reducing main channel stages. At Rulo, NE, the stage reduction was 2.6 feet. Stage reduction in this area was influenced by levees which were <u>raised</u> from their existing elevation to the 25 year level.

16.3.2. Peak Discharge. Within the federal levee reach, peak discharge reduction was substantial and varied from 10,000 cfs at Plattsmouth (-6.5%) to 60,000 cfs near the downstream end of the federal levees at RM 520 (-21.0%). As with stage, the effect of storing additional water is clearly

illustrated by the peak discharge reduction.

16.3.3. Hydrograph Timing. The time at which peak stages occurred throughout the reach was altered by the numerous levee failures. As a result of the hydrograph timing shift, different inflow conditions would produce different results. Effects of timing are illustrated by the peak discharge at Nebraska City, NE. Due to the levee failure timing combined with timing of inflow from the Platte River and Weeping Water Creek, the peak discharge increases compared to the upstream and downstream reported stations. At the Nebraska City, NE location, the time at which peak values occurred was altered from the base condition.

16.3.4. Levee Cells. A negative aspect of setting all levees at the 25 year level is that significantly more areas were damaged. Of the 12 levee cells contained within the UNET model, all but 2 cells were overtopped and flooded. Maximum stage depths within the 10 flooded cells average 7.5 feet. Failure of the 10 levee units removes flood protection from approximately 160,000 acres. During the 1993 event, only one federal levee failed and flooded two UNET model interior cell areas. Flooding an additional 9 levee cells may not justify the reduction in the hydrograph observed at Rulo, NE.

16.4. Without Federal Reservoirs.

As expected, increasing the inflow to the study reach increased computed stages and discharges. The increase varied throughout the reach depending on the effects of levee failures, inflow hydrographs, and etc.

16.4.1. Peak Stage. Removal of the federal reservoirs caused a peak stage increase between Gavins Point and Brownville, NE. Between Omaha, NE and Brownville, NE, the stage increase changed from 4.9 feet to 0 feet. As a result of higher stages, additional federal levees overtopped and failed in the reach between Omaha, NE and Brownville, NE which attenuated flow. Therefore, the effect of increased flow was mitigated by additional levees failing.

16.4.2. Peak Discharge. Within the federal levee reach, the peak discharge increase was substantial with a net change of over 65,000 cfs at Omaha, NE (47.9%). However, a reduction of 15,000 cfs near the downstream end of the federal levees near RM 520 (-5.5%) was experienced. The peak discharge reduction was based on the additional federal levees overtopping and failing between Omaha, NE and Brownville, NE. This attenuated the flow and altered the timing of the peak discharge causing an overall reduction in peak flow at RM 520 compared to the computed hydrograph.

16.4.3. Hydrograph Timing. The time at which peak stages occurred throughout the reach was altered by the numerous levee failures. As a result of the hydrograph timing shift, different inflow conditions would produce different results.

16.4.4. Levee Cells. Compared to the 1993 event when two cells were flooded, the without upstream federal reservoir alternative resulted in flooding 7 of the total 12 levee cells between Omaha, NE and Rulo, NE. Stages within the flooded cells were also significantly higher than the 1993 base and 25 year level alternative.

16.5. 5 and 10 Percent Hydrograph Reduction.

Study results generally confirmed that reducing inflow will reduce stage and discharges.

Achieving the computed results requires that runoff reduction does not alter the timing of peak flows and levee failure.

16.5.1. Peak Stage. Peak stages were reduced by a relatively insignificant amount of less than 1 foot throughout the routing reach for the 5 percent reduction. Further reducing hydrographs by 10 percent provided an additional stage reduction of 0.5 to 0.8 feet within the Omaha, NE to Rulo, NE reach. For the 10 percent alternative, stage reduction varied from a minimum of 0.5 feet at Rulo, NE to a maximum of 1.5 feet at Nebraska City, NE.

16.5.2. Peak Discharge. Peak discharge reduction throughout the reach varied at each location with an average generally equal to the original 5 or 10 percent applied to the inflow hydrographs. At river mile 520, a peak discharge of 273,000 cfs for the 10 percent reduction alternative actually exceeded the peak discharge of 272,000 cfs for the 5 percent reduction alternative. Discharge reduction was affected by the amount of water stored within levee cell L-550. For the 10 percent alternative, discharge reduction varied from a minimum of 29,500 cfs at RM 520 (-9.7%) to a maximum of 23,200 cfs at Nebraska City, NE (-11.4%).

16.5.3. Hydrograph Timing. Timing was altered between the 5 and 10 percent reductions. Within table OM-10 and OM-11, comparison of peak stages and discharges at Brownville, NE and RM 520 illustrate fluctuations. The departure in results from the trend apparent at other locations is due to the combination of L-550 levee failure time and inflows from the Little Nemaha River. At the Brownville, NE and RM 520 locations, the time at which peak values occurred was altered from the base condition.

16.5.4. Levee Cells. Applying both the 5 and 10 percent hydrograph reduction did not reduce stages enough to prevent the failure of federal levee L-550. For the 5 percent reduction alternative, the peak stage was reduced by 1.0 feet in the L-550 upper cell and 4.1 feet in the L-550 lower cell when compared to the computed stages. For the 10 percent reduction alternative, peak stage was reduced by 1.4 feet in the L-550 upper cell and 6.1 feet in the L-550 lower cell.

16.6. Isolated Levee Setback - Case Study.

Results from the setback alternative illustrated the undesirable effect of causing downstream impacts while providing beneficial results to the local area. The levee setback from RM 542.1 to RM 516.3 reduced stages within the setback reach but increased downstream Missouri River stages for a distance in excess of 65 miles.

16.6.1. Peak Stage. Peak stages dropped at both Brownville, NE and RM 520 by 1.9 and 0.9 feet, respectively, when compared to base condition computed stages for the same locations. However, the stage at Rulo, NE was 1.0 feet higher then the computed peak stage. Computed stage increases where also illustrated downstream at St. Joseph, MO (RM 448.2) where an increase of 0.4 feet was determined.

16.6.2. Peak Discharge. With the setback, discharges increased about 7900 cfs at Brownville, NE. This is about a 3.3 percent increase over the computed peak discharge at the same location. Peak discharge increase at RM 520 was only about 1.5 percent. At St. Joseph, MO (RM 448.2), a 1.4 percent peak discharge increase was still computed by the model.

16.6.3. Hydrograph Timing. The levee setback condition did not appear to significantly

alter the hydrograph timing from the 1993 computed condition.

16.6.4. Levee Cells. Federal levee L-550 was prevented from overtopping and failing. Therefore, no storage cells within the Omaha District were overtopped for the levee setback alternative. However, stages within the Rulo overbank area and levee cells downstream of Rulo, NE within the Kansas City District were increased.

16.7. Levee Setback - Systemic Study.

Results from the systemic setback alternative again illustrates the undesirable effect of causing downstream impacts while providing beneficial results to the local area. The levee setback starting at RM 600.0 reduced stages within the reach to Rulo, NE (RM 498.0). However, Missouri River stages were increased downstream of Rulo, NE. For the levee setback that contained the 1993 flood event with no levee failures, stages were reduced from RM 600.0 to Nebraska City (RM 562.6) but increased downstream from there.

16.7.1. Peak Stage. For the setback alternative with the existing levee top elevations, peak stage reduction ranged from -0.4 feet at Plattsmouth, NE to -1.4 feet at Nebraska City, NE. For Rulo, NE, there was an overall increase in stage of 1.0 feet because the setback of the levees altered the failure of the private levees. For the setback alternative that confined the 1993 flood event with no levee failures, only Plattsmouth, NE and Nebraska City, NE had reductions in stage of -0.4 and -1.4 feet, respectively. Brownville, NE, Rulo, NE and St. Joseph, MO all experienced an increase in stage. The increase was due to no failures of federal levee L-550 and the Rulo Overbank area.

16.7.2. Peak Discharge. With the setback, peak discharges ranged from an increase of about 36 percent at Rulo, NE to a decrease of about 4.4 percent at Brownville, NE and 6.5 percent at RM 520. The increase at Rulo was caused by moving the levees back away from the Missouri River channel edge. This allowed for a larger flow to pass through the river channel before the Rulo Overbank levees failed. The decrease at Brownville, NE and RM 520 was caused by the altering of the time at which federal levee L-550 failed. This altered the hydrograph enough to change the timing of the peak discharges at the two locations. At St. Joseph, MO (RM 448.2), a 1.7 percent peak discharge increase was computed by the model. For the setback alternative with the 1993 flood confined, discharges at most stations were similar to the alternative that contained the 1993 flood event but without the levees being setback a minimum of 5,000 feet.

16.7.3. Hydrograph Timing. The time at which peaks occurred was slightly altered as described in the peak discharge section at St. Joseph and Rulo. Upstream of Rulo, the levee setback condition did not appear to significantly alter the hydrograph timing from the 1993 computed base condition.

16.7.4. Levee Cells. For the setback, federal levee L-550 still overtopped and failed. However, stages in the overbank were reduced by 0.6 feet for L-550 UP and 2.7 feet for L-550 DN when compared to the computed base condition stages.

16.8. Results Summary.

The performed analysis illustrates that no single alternative provides beneficial results throughout the system. Applying a single policy system wide will cause undesirable consequences at some locations.

Several of the alternatives altered hydrograph timing. A complete evaluation is required prior to implementing any alternative to investigate performance for a variety of events with different inflow characteristics. Alternatives which provide a local beneficial impact by reducing flows and stages may cause downstream consequences when the timing of levee failures and hydrograph peaks is altered.

Understanding results and the effects of each alternative requires the comparison of computed peak stages, discharges, and levee cell stages shown in tables OM-10 through OM-12. All of these variables illustrate how an alternative affects performance of the flood control system as a whole. For example, the levee setback and fully confined alternatives reduces the flooded areas. However, an undesirable side effect is that peak stages and discharges at downstream locations are increased. Results of the levee removal alternative illustrated that all model results which determine a stage and discharge reduction are extremely dependent upon assumptions regarding floodplain use and flow roughness. Results of the 25 year notch and runoff reduction alternatives illustrated that timing of levee failure combined with tributary inflows altered the time at which peak stages and discharges occurred.

The runoff reduction, levee removal, and 25 year notch alternatives all reduced computed peak flow and stage. Reductions for the levee removal and 25 year notch alternatives were possible as the result of the inundation of a significant area between Omaha, NE and Rulo, NE. Runoff reductions would also require the additional inundation of a significant area. Comparing at the downstream model limit, both the levee removal alternative and the 25 year notch alternative provide discharge and stage decreases which exceed the decrease computed for the runoff reduction alternatives. Within the Omaha District, reductions from the 25 year notch alternative appeared to equal or exceed reductions from the levee removal alternative.

17. Developed Products and Noted Deficiencies.

Several important products were developed during the course of the FPMA study which have numerous potential uses for agencies involved with the Missouri River floodplain. Several inadequacies were also noted during the course of the study which should be addressed. A brief summary of a few of the more prominent study products and noted deficiencies is as follows:

- A digital terrain model of existing topography was developed from scanned quadrangle topographic maps for the Missouri River and floodplain from Rulo, NE to Omaha, NE. The digital terrain model provides a means of extracting topographic information along the Missouri River for numerous uses.

- The UNET model developed for evaluation of the 1993 flood event and alternatives provides an extremely valuable tool. Employing the UNET model allows the evaluation of proposed modifications to examine system wide effects. Events other than 1993 may be modeled to investigate system performance and highlight problem areas.

- Digital flood boundary mapping capability was developed to allow the rapid assessment of flood boundaries and inundation depths for various water surface elevations along the Missouri River.

- Existing information on levee quality, elevation, and spatial location is sparse. Identifying levee features is required to provide an accurate assessment of the protection provided by the levee system. The extent of constructed private levees is relatively unknown. The private levee system significantly impacts the performance of the federal levee system.

• Geometry employed for creation of the UNET model is in excess of 20 years old for the Missouri River. Tributary geometry data is generally unavailable and was extracted from USGS quadrangle topographic maps for UNET model input. Sedimentation effects on Missouri River and tributary geometry are known to be extensive during the past 20 years. Updating model geometry is required to accurately assess impacts of current and future flood events.

18. Conclusions.

Simulation of the 1993 event and the various alternatives illustrated several positive and negative aspects of floodplain management. The FPMA study focused on the 1993 event only. Other events may generate different conclusions. Applying what appears to be good floodplain strategy within a limited area can have undesirable effects at other locations within the river system. Employing an unsteady flow model to simulate the 1993 event and alternatives illustrated that the entire system must be evaluated as a whole and not in individual segments. Several of the alternatives examined showed potential for decreasing damage associated with an event similar to 1993.

General

• All study computations were performed for the 1993 event only. Extrapolating conclusions obtained from analysis of 1993 event modeling may be erroneous with respect to other events. For example, determining whether any individual levee cell will fail varies for each alternative and flood event. An individual cell may or may not fail depending upon the Missouri River flow, tributary inflow, and levee failure either upstream or downstream of the individual cell. Levee failure parameters also vary including time of failure, computed flow, and ponding depth and duration within the cell.

• Study results proved that a system-wide hydraulic analysis is required to properly evaluate alternative projects rather than looking at each independently. Basin wide planning is required to completely evaluate effects of proposed alternatives along the Missouri River. Future levee and floodplain development must be evaluated on a system wide basis employing an unsteady flow model.

• Several of the alternatives altered hydrograph timing. A complete evaluation is required prior to implementing any alternative to investigate performance for a variety of events with different inflow characteristics.

• Several previous studies conducted by Omaha District on the Missouri River system between Omaha, NE and Rulo, NE have documented sediment deposition and a reduction in conveyance within the reach. Future trends analysis indicates that the level of protection provided by the federal levee system will continue to decrease.

• Modeling and alternative analysis performed for the FPMA is of initial assessment level of detail. Additional study is required to evaluate base and alternative condition impacts on infrastructure, interior drainage, flood duration, homes, livestock, flood fighting, and etc.

• Previous studies and the 1993 event illustrated that the agricultural federal levee system within the Omaha District does not provide the original 50 year level of protection. Test cross sections were surveyed in 1995 and compared to 1974 data along the Missouri River. Employing section conveyance to estimate the affect on water surface, the 1974 section conveyed the same amount of flow at an elevation

2 to 3 feet lower than the current section. Results indicate a continuing decrease in the level of protection provided by the federal levee system within the Omaha District.

Agricultural Levees

• Modeling results demonstrated that agricultural levee removal does not provide uniform stage and discharge reduction. Depending on floodplain usage, peak discharge reduction may not be accomplished by levee removal. Within the levee removal area, river stage is reduced but the flow reduction is much less. As a result, higher flows may be routed downstream to critical and urban areas where the levees were not removed. Higher flows at the confined locations, which correspond with the high damage locations, will cause increased stages at these locations.

• Modeling results demonstrated that agricultural levee removal does provide stage reduction. The minimal average stage reduction of 1 - 3 feet may not justify removing flood protection from an extremely large area.

• Agricultural levee removal modeling was performed with a high and low loss value to encompass a range of floodplain conveyance. Variation in computed stage between the two floodplain conditions varied from 2 - 3 feet. Factors such as effective flow width, channel meandering, induced sediment deposition, vegetation, land use, topography, structures such as roads and railroads, and other conveyance parameters are all combined within the single high or low loss value. The low loss value levee removal alternative provides the maximum stage reduction. Low loss floodplain conveyance assumes that all major infrastructure encroachments are removed, the vegetative growth is kept to a minimum, no sediment deposition is allowed within the slack water overbank regions, and etc.

• Agricultural levee removal low and high loss values also illustrated discharge reduction for both alternatives compared to the base condition. The high loss floodplain condition, with environmental habitat areas, produced higher stages than the low loss condition but provided additional discharge reduction. Results from the levee removal high loss condition showed an additional average peak discharge reduction of 5 - 10% compared to the low loss condition.

• Currently, areas between the Federal levee and the Missouri River have agricultural usage in most areas. Setbacks or levee removal will not provide additional environmental habitat without regulation and enforcement.

• Providing a 25 year elevation levee succeeded in reducing stages and flows since floodplain storage was increased. However, significant areas which were not flooded in the base 1993 event were flooded in the alternative as a result of the reduced level of protection. Designating specific areas for overtopping may be feasible. However, an extremely large area is required in order to store the volume required to impact the flood hydrograph.

• Although the 25-year elevation levee alternative appeared to have positive results on reducing Missouri River stages, it is unknown if the project would function as modeled during an actual flood event. Rigorous basin wide analysis for a full range of events is required to determine optimum levee overtopping and designed failure features. In addition, it is likely that local residents would attempt to flood fight the levee notch areas and eliminate the attenuating effects of the notches.

• The levee setback case study illustrated that setbacks of a particular Omaha District federal levee would have prevented failure of that levee during the 1993 event. However, levee setbacks were also shown to have undesirable consequences. If levee setback distance is sufficient such that the levee no longer fails, results showed that a downstream rise in flow and stage is caused. It is also likely that increased vegetative growth between the levee and river would increase roughness and offset the effects of the levee setback. In addition, negative impacts to interior drainage would include a longer outlet channel to discharge into the river requiring increased maintenance due to siltation and etc.

• The confined condition alternative where levees were raised to confine all flow to the Missouri River did not significantly impact computed stages and discharges within the federal levee area of the Omaha District. Within the Rulo, NE area, where the private levee system suffered extensive damage in the 1993 event, confinement caused a stage increase of nearly 8 feet.

• Significant levee areas were flooded in the 1993 event by interior ponding and not actual levee failure or overtopping. The interior drainage study determined that pumping requirements to prevent all damage within the levee areas for the 1993 event is prohibitive. A detailed interior drainage analysis is required for all alternatives examined to determine the optimum size and placement of interior drainage features.

• Interior ponding levels are affected by factors such as rainfall, runoff into the cell from contributing drainage areas, seepage, and the peak stage, timing, and duration of Missouri River hydrographs. The interior drainage study determined that the alternatives examined would not cause a significant variation in ponding levels from the base condition.

Watershed Measures

• Comparison of the no reservoir alternative and base condition illustrates the stage reduction provided by the Missouri River mainstem reservoirs. Results of the no reservoir alternative determined stage increases which varied from 0 to 5 feet between Omaha, NE and Rulo, NE. Many additional levee cells were overtopped and areas flooded.

• Watershed storage measures were modeled as 5% and 10% percent hydrograph reduction. At Rulo, NE, a 10% volume reduction equates to 1.7 million acre-feet. Compared to the large amount of volume removed from the hydrograph, relatively insignificant reductions of 1 - 1.5 feet were observed in peak stages within the Omaha District.

• An extended drought occurred in the upper Missouri River basin prior to 1993 with mainstem reservoir levels below normal. Releases during the 1993 event from the lowest reservoir (Gavins Point) were minimal. An analysis was performed to evaluate releases if antecedent conditions in the upper Missouri River Basin had been at normal levels and the mainstem reservoir level in 1993 had been at an average end of month elevation for May. Although operation procedures may have varied slightly, analysis determined that the excess inflow into Gavins Point would have been less than the volume released during the actual 1993 operation of the reservoirs. Therefore, additional releases in 1993 of a magnitude which would have significantly impacted 1993 flood levels would not have been required if initial pool levels had been closer to average.

• If antecedent conditions had been such that only the exclusive flood control zone were available

in the mainstem Missouri River Reservoirs, analysis determined that approximately one third of the inflow would have been captured by the reservoirs. However, reservoir pool levels within the exclusive flood control zone at the end of May has not occurred at anyone of the six dams and should be regarded as an extremely rare event. Following normal operation procedures, actual 1993 operation captured approximately 80% of the inflow. Although capacity to store near 100% of the inflow was available, minimal releases during the summer of 1993 were necessary for downstream water uses. The no reservoir alternative assumed 0% capture of inflow. Reservoir storage effects on discharge reduction is bracketed between computed results for the base and the no reservoirs alternative.

19. Recommendations.

The 1993 flood event, previous studies, and the FPMA analysis of the base condition and alternatives illustrated several areas of concern with the existing floodplain management and protection system. Based on results of the FPMA and other studies performed by the Omaha District, the following recommendations target specific concerns:

General

- Hydrology and hydraulics for the Missouri River system should be updated to provide an accurate assessment of 100 year flood boundaries and the level of protection provided by the current system. Current discharge-frequency curves have been shown by brief studies to be significantly in error. Geometry updating of hydraulic model data for the Missouri River and tributaries is also required. Ramifications of updating discharge-frequency curves and model geometry are extensive and include revising current FIS (designated floodways, 100 year flood boundaries, etc.) for the entire Missouri and Mississippi Rivers. Continued periodic updating of hydraulic and hydrologic models is recommended to reflect changes caused by sedimentation, land use, basin development, hydrologic effects, and etc.

- The FPMA study should be expanded to investigate performance of the base and alternative conditions for events other than 1993. Evaluating other events is required to determine flood control system performance, highlight problem areas, and reveal potentially serious flaws in the current flood control system and proposed alternatives.

- The Omaha District has developed numerous emergency system operating plans and dam failure contingencies based on outdated models for the Missouri River between Gavins Point and Rulo, NE. Studies within this area should be updated employing the UNET model developed for the FPMA.

- An increased level of coordination between Corps Districts and elements within each District is required to provide evaluation of projects which have basin wide implications. Past history indicates that Hydrology & Hydraulics personnel are often not included in evaluation of many scenarios which directly involve aspects of floodplain features of the Missouri River.

Levee Systems

- An evaluation of levee failure scenarios is recommended. For large levee units, analysis and design can be employed to limit interior damage areas and prevent successive levee failures. Failure at the upstream end of a levee cell produces large ponding depths at the downstream end of the levee cell

and may fail adjacent cells. During the 1993 event, the tendency of levee failures to cascade downstream was illustrated at several locations.

• Previous studies and the 1993 event illustrated that the agricultural federal levee system within the Omaha District does not provide the original 50 year level of protection. Measures should be taken to reduce potential liability during an event such as 1993.

• Test cross sections were surveyed in 1995 and compared to 1974 data along the Missouri River. Employing section conveyance to estimate the affect on water surface, the 1974 section conveyed the same amount of flow at an elevation 2 to 3 feet lower than the current section. A complete hydraulic analysis to determine the effect on water surface elevation as a result of the change in section geometry was not performed.

• Evaluation of tributary levee tiebacks capacity is required. Sedimentation within the tieback system has severely reduced capacity of tieback levee systems and increased the flood threat.

• A intensive basin wide study should be conducted to determine which private levees should be removed and which private levees may be beneficial. Previous studies and observations indicated that private levees which are riverward of the constructed federal levees reduce the level of flood protection provided by the federal levee system.

• Examination of employing federal funds to repair levees systems is required. Following the 1993 event, several Federal agencies and programs were available to provide assistance with levee repair. Repair of levee systems was not investigated with any hydrologic models. Within the Omaha District, numerous private levee systems were repaired with federal funds. Studies indicate that some private levees are detrimental to flood protection provided by federal levees and contribute to erosion damage, higher stages, and increased sediment deposition. Repair of private levees was promoted by federal agencies even though these same levees compromise the effectiveness of the federal levee system.

• Levee profile surveys of all federal levees should be obtained. The most recent profiles within the Omaha District are 40-50 year old as-built surveys in most cases.

• An inventory of all private levees and profile surveys should also be obtained. Top priority should be given to private levees situated riverward of federal levees on both the main river and tributary tiebacks.

• Alternative methods of reducing sedimentation impacts on the berms and tributaries should be investigated.

• Interior drainage structures should be inventoried and a data base created with pertinent data for each structure. Ponding areas should be identified on topographic maps.

• Most levee areas experienced flood damage in the 1993 event as a result of inadequate or poorly functioning interior drainage facilities. Problems with interior drainage facilities include sediment deposition, erosion, and deterioration of the structures since construction. A comprehensive study is recommended to assess the discharge capability, adequacy of the facilities compared to the 1993 event, and current as well as future structural condition of interior drainage features.

Floodplain Management

• Following the 1993 flood event, FEMA flood boundaries and actually observed flood boundaries should be compared. Within the Omaha District, current FEMA 100 year floodplains were developed from data in excess of 20 years old. An active program to periodically update FEMA flood boundaries should be developed to identify all areas at risk. Currently, revisions often appear to occur in response to an extreme flood event such as 1993 rather than before the event based on good engineering practice.

• A coordinated floodplain policy between state, federal, and local governments is required. Different jurisdictions and policies have resulted in the regulation of floodplains and floodways which varies considerably from one location to another.

• Revisions to current zoning and development policies is recommended. Local interests often neglect hydrologic impacts on adjacent areas. A basin wide planning and review authority is an alternative to provide increased coordination.

• A flood warning system for leveed areas which are subject to failure and inundation is recommended to prevent loss of life and reduce damages.

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FPMA

FloodPlain Management Assessment

Hydrology and Hydraulics

Kansas City District

May 1995
Final Report

US Army Corps
of Engineers

FLOODPLAIN MANAGEMENT ASSESSMENT OF THE UPPER MISSISSIPPI RIVER AND LOWER MISSOURI RIVER BASINS
U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT

HYDROLOGY AND HYDRAULICS FINAL REPORT
MAY 1995

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UPPER MISSISSIPPI RIVER AND LOWER MISSOURI RIVER BASINS

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT

Hydrology and Hydraulics Final Report
May 1995

I. INTRODUCTION

In the aftermath of the Great Flood of 1993, many questions were raised regarding the impact of environmental factors and structural systems on flood stages. Examples of the topics broached include land use in the floodplain, and the effects of various components of the basin such as levee systems, wetlands in the upper river basins, and reservoir systems and their operation. In order to examine some of these questions in a formal and systematic manner, Congress authorized the Floodplain Management Assessment (FPMA) as a comprehensive study of the Missouri and Mississippi River Basins, and how they reacted to the conditions imposed by the Great Flood of 1993.

The mathematical computer model program UNET, developed and programmed by Dr. Robert Barkau, was chosen to simulate the river system as the hydraulic basis of this study. UNET is a one-dimensional, unsteady flow program which simulates unsteady flow through a full network of open channels and reservoirs (Reference No. 2). UNET models were developed for reaches of the rivers based on the location of USGS rated gauges with respect to Corps of Engineers district boundaries. The data from these gauges were used to establish upstream and downstream boundary conditions for each UNET model. U. S. Army Corps of Engineers (Corps) districts involved with the UNET modeling effort are Omaha and Kansas City on the Missouri River, and Rock Island and St. Louis on the Mississippi River.

For the purposes of this study, the Kansas City-Missouri River UNET model was developed, calibrated, and used, in conjunction with the UNET models developed by the other aforementioned districts, to conduct several system-wide hydraulic analyses, hereinafter referred to as alternatives. These alternatives entailed hypothetical, uniform, system-wide changes in the structures and operation of the Missouri/Mississippi basin, and are designed to analyze the impact of these changes on 1993 flood flow conditions. The following alternatives were simulated with the UNET model program:

Alternative No. 1. Existing 1993 flood conditions, also called base conditions.

Alternative No. 2. Removal of all agricultural levees with an agricultural regime in the floodplain.

Alternative No. 3. Removal of all agricultural levees with natural ecological succession in the floodplain.

Alternative No. 4. Raise all levees to completely contain the 1993 flood flows.

Alternative No. 5. Alter the height of all agricultural levees to correspond to the 25-year frequency water surface profile.

Alternative No. 6. Reduce 1993 runoff volume by 5%.

Alternative No. 7. Reduce 1993 runoff volume by 10%.

Alternative No. 8. Existing levee conditions with 1993 computed natural inflow hydrographs, i.e. 1993 flood discharge hydrographs computed to simulate the absence of federal reservoirs.

Alternative No. 9. Set agricultural levees back from their existing location to create a "setback" floodway of 1.5 times the existing floodway width or 5000 feet wide, whichever is greater.

II. STUDY AREA DESCRIPTION

A. Basin Description

The Missouri River originates in the northern Rocky Mountains at Threeforks, Montana, and flows south and east for 2,315 miles to join the Mississippi River at a point approximately 15 miles above St. Louis, Missouri. The Missouri River Basin, which drains 74 percent of the upper Mississippi River Basin, is the largest of the United States' major water resource regions, embracing 513,000 square miles within the United States and 9,715 square miles in Canada. Hydrologically, the Missouri River Basin is divided into upper and lower portions with demarcation at Sioux City, Iowa. The lower Missouri River Basin contains 214,700 square miles. (Reference Nos. 17, 18, & 19).

The drainage area within the Kansas City District amounts to 110,445 square miles. The Kansas City District's portion of the Missouri River Basin includes all of the lower basin drainage area from river mile 498.1 at Rulo, Nebraska, to the mouth near St. Louis. The Omaha District has responsibility for all of the Missouri River Basin upstream of the Kansas City District, i.e. the entire drainage basin upstream of Rulo to the headwaters. From Rulo to Kansas City, the Missouri River flows through the dissected till plains of the central lowlands. Downstream of Kansas City, the river flows along the northern border of the Osage Plains and the Ozark Plateau to a point near St. Charles, Missouri, where it re-enters the central lowlands to join the Mississippi River. (Reference No. 18).

Between Rulo, Nebraska, and the mouth at St. Louis, the Missouri River has a total fall of about 451 feet and the average slope varies from 0.8 to 1.0 foot per mile. The river within this reach contains approximately 865 miles of bankline in Missouri, 140 miles in Kansas, and eight miles in Nebraska. The fringe area along the river is covered with willows and other trees. The floodplains are comparatively wide and for the most part are under cultivation (Reference No. 20). The width of the floodplain varies from a maximum of approximately thirteen miles to a minimum of approximately 1.5 miles. The actual flow way decreases to less than 0.5 mile in reaches with urban levees at Kansas City and St. Charles.

The Missouri River contributes 42 percent of the long-term average annual flow of the Mississippi River at St. Louis and is the major contributor of sediment in the upper Mississippi River Basin (Reference No. 17). U. S. Geological Survey (USGS) measurements from the first wave of the 1993 Flood indicate that an average of eight to ten feet of scour had occurred from St. Joseph to Hermann, Missouri. The USGS made estimates of the volume of suspended sediment that passed the gauge at Hermann between June 26 and September 14, 1993. Total suspended sediment volume was estimated to be 76.8 million tons, which included 21.8 million tons of sand and 55.0 million tons of silt and clays (Reference No. 18).

B. Flood Control Structures

The Missouri River Basin contains numerous reservoirs and impoundments. The Corps of Engineers has constructed six mainstem Missouri River Dams which are all located upstream of Rulo, Nebraska, and are within the boundaries of the Omaha District. All reservoirs within the Kansas City District are constructed on tributaries of the Missouri River. These include eighteen multiple-purpose lake projects constructed by the Corps and eleven lake projects constructed by the Bureau of Reclamation (Bureau). The eleven Bureau lake projects are all in the Kansas River Basin. The Bureau operates these lake projects primarily for the storage and distribution of water for irrigation, while the Corps is responsible for the flood control operation of the Bureau's lakes as part of the lower Missouri River flood control system (Reference No. 18).

During the 1993 Flood, federal levee units performed as designed, but damage occurred since the duration and magnitude of the flood exceeded the design criteria. The Kansas City District has constructed fifteen levees as part of the Missouri River Levee System (MRLS). All but four of the completed MRLS units are upstream of Kansas City. Six of the MRLS units were substantially overtopped resulting in four of them being completely breached by scour and erosion of the levee embankment. Some levees which had been designed with two to five feet of freeboard did not overtop even though their design discharge was exceeded. Some of the MRLS units which were not overtopped experienced interior damages from heavy seepage flows and ponding of interior runoff (Reference No. 18).

Non-federal levees along the Missouri River were devastated by the 1993 Flood. Approximately 99 percent of all non-federal levees were breached from Brownsville, Nebraska, to the mouth, which is a distance of 535 river miles. Failures occurred by breaching, overtopping, wavewash, sidewash, and topwash. At the peak of the flood, all non-federal levees in the lower reach of the river were completely inundated and the floodplain functioned as if the levees did not exist. Many deep scour holes were formed in the floodplain and large quantities of sediment were deposited, as much as eight to ten feet deep at many locations (Reference No. 18).

C. Description of Major Tributaries

Most of the major tributaries of the Missouri River are included as separate routing reaches in the UNET model. Inclusion depends on the existence of USGS gauging stations on the tributaries. All major tributaries entering the Missouri River are modeled from the last downstream rated gauge on the tributary to the confluence with the Missouri River, in order to reproduce the effects of backwater on the outflows (Reference No. 1). Major tributaries of the Missouri River which have the requisite gauging are the Big Nemaha, Nodaway, Platte, Kansas, Big Blue, Little Blue, Grand, Chariton, Little Chariton, Blackwater/Lamine, Osage, and Gasconade Rivers. A schematic of the Missouri River and its significant tributaries included in the Kansas City-Missouri River UNET model is illustrated on Plate KC-1. Also on this plate are the locations of the gauges on the tributaries and on the mainstem.

1. Big Nemaha River - RM 494.9

The Big Nemaha River is a right bank tributary of the Missouri River that drains 1920 square miles in southeastern Nebraska and northeastern Kansas, of which 1315 square miles lie in Nebraska. The topography of the basin consists of gently rolling to steeply rolling hills drained by a dendritic stream pattern. The major streams of the basin have wide flat floodplains which are generally poorly drained. Stream valley soils consist of alluvial and colluvial materials which are easily eroded.

Basin elevations range from about 840 feet N.G.V.D. at the mouth of the Big Nemaha River to a maximum of 1535 feet N.G.V.D. Stream slopes vary from 2 feet per mile in the lower reaches to over 20 feet per mile on some tributaries of the Big Nemaha River. Extensive channel modifications have increased stream slopes and caused channels to deepen and widen progressively upstream. Stream flow in the basin is due almost solely to runoff from precipitation and consequently shows frequent and wide fluctuation. There no major impoundments in the Big Nemaha River basin. (Reference No. 5).

2. Nodaway River - RM 463.0

The Nodaway River is a left bank tributary of the Missouri River that rises in the low, flat divide of southwest Iowa between the Missouri and Mississippi River basins. It flows southwesterly through Iowa, and then southerly through the northwest corner of Missouri to its confluence with the Missouri River. The main stem of the Nodaway River is a little over 61 miles from its mouth to the confluence of West Nodaway and East Nodaway Rivers. Continuing upstream on Middle Nodaway to its headwaters results in a total river length of about 130 miles.

Channel slopes vary from a relatively flat two feet per mile in Missouri to six feet per mile in the upper reaches of Iowa. The average channel slope is four feet per mile. The Nodaway River is considered to be a small stream with a relatively low average discharge of 524 cfs despite its relatively large drainage area of 1780 square miles. Approximately 67%, or 1200 square miles, of the drainage basin lie in Iowa. There are no major impoundments in the Nodaway River basin. (Reference No. 14).

3. Platte River - RM 391.1

The Platte River is a left bank tributary of the Missouri River that rises in the low, flat divide of southwest Iowa. It flows in a generally southerly direction through Iowa and Missouri. The main stem of the Platte River is formed by the confluence of the East Platte and West Platte Rivers in Iowa and is approximately 170 miles in length from the confluence to the mouth. River slope for the Platte River ranges from one to three feet per mile in Missouri and increases in the upper reaches in Iowa.

The Platte River basin drains an area of 2440 square miles, of which 32% is in Missouri and 68% is in Iowa. The basin lies generally north and south with approximate dimensions of 130 miles in length and 27 miles in maximum width. The topography of the basin consists of rolling or gently undulating glacial plains divided by deeply eroded valleys. The major impoundment in the basin is a Corps reservoir, Smithville Lake, which is on the Little Platte River, a major tributary of the Platte River. (Reference No. 16).

4. Kansas River - RM 367.5

The Kansas River is a right bank tributary of the Missouri River that is formed at the confluence of the Smoky Hill and Republican Rivers near Junction City, Kansas. From this junction the river flows eastward for about 170 miles to its confluence with the Missouri River at Kansas City. Locally, the main stem Kansas River is also known as the Kaw River. The floodplain of the Kansas River from Junction City downstream varies in width from approximately 1.5 to 5.0 miles and averages approximately two miles in width. The channel, which is generally 800 to 850 feet wide, meanders in this floodplain.

The entire Kansas River drainage basin lies within the Interior Plains region and is approximately 480 miles long and 140 miles wide. Elevation of the river varies from 750 feet N.G.V.D. at the mouth to

approximately 5500 feet N.G.V.D. at the extreme western end of the basin. Channel slopes west of Concordia, Kansas, are approximately 12 feet per mile. The average channel slope downstream of Topeka, Kansas, is approximately 1.7 feet per mile.

The Kansas River basin constitutes approximately one-tenth of the drainage area of the Missouri River and drains the northern half of Kansas, much of southern Nebraska, and a part of northeastern Colorado. The total drainage area of the Kansas River basin is 60,060 square miles of which 15% is in Colorado, 28% is in Nebraska, and 57% is in Kansas. The Kansas River basin contains numerous major impoundments including seven Corps reservoirs and eleven Bureau of Reclamation reservoirs. (Reference Nos. 4, 23, & 24).

5. Big Blue River - RM 358.0

The Big Blue River is a right bank tributary of the Missouri River which is formed by the confluence of Coffee Creek and Wolf Creek in west-central Kansas and flows north-northeasterly into Missouri to its mouth in the eastern Kansas City urban area. The Big Blue River is 43.8 miles long and drains a basin that encompasses a total area of 272 square miles. Approximately 56% of the basin lies in Kansas and 44% lies in Missouri.

The Big Blue River basin measures approximately 31 miles in length and 17 miles at its maximum width. The topography of the basin is predominately rolling to gently undulating with fairly steep slopes adjacent to the larger streams. There are numerous channel improvement projects scattered throughout the length of the Big Blue River. The river channel slope ranges from 3 to 12 feet per mile with an average of 5 feet per mile. There are no major impoundments in the basin. (Reference No. 9).

6. Little Blue River - RM 339.5

The Little Blue River is a right bank tributary of the Missouri River which rises in west-central Missouri and flows in a generally northeasterly direction to join the Missouri River about 20 miles downstream of Kansas City. The main stem of the river is about 50 miles in length. The Little Blue River basin lies along the southeastern edge of the Kansas City metropolitan area and drains an area of 224 square miles. The basin is approximately 33 miles long, with a maximum width of 13 miles.

The topography of the basin is predominately rolling to gently undulating. The streams of the upper basin originate in steep terrain and limestone outcrops, and have well-incised channels. The slope of the lower basin averages 2 to 3 feet per mile, and steepens rather abruptly to 13 or more feet per mile in the upper reaches. Major impoundments in the basin include two Corps reservoirs and Lake Jacomo, a Jackson County (Missouri) public recreation lake. (Reference Nos. 8 & 13).

7. Grand River - RM 250.0

The Grand River is a left bank tributary of the Missouri River that rises in the low, flat divide of south-central Iowa and flows generally in a south-southeasterly direction. The topography of the Grand River basin ranges from rolling to gently undulating glacial plains divided by deeply eroded valleys. The Grand River basin drains an area of 7900 square miles, of which 78 percent is in Missouri and 22 percent is in Iowa. The main stem of the Grand River is about 210 miles in length, which includes the West Fork as part of the main stem. The slope of the river ranges from 1.0 to 2.0 feet per mile up to mile 148, with an average slope of 1.5 feet per mile.

Tributaries of the Grand River include the Thompson River with its major tributary the Weldon River, and numerous small tributaries, many of which are intermittent in character. The Grand River has undergone extensive channel modifications which were primarily projects of local drainage districts and other private interests. There are no major impoundments in the basin. (Reference No. 12).

8. Chariton River - RM 238.8

The Chariton River is a left bank tributary of the Missouri River that rises in the low, flat divide of south-central Iowa. It flows southeasterly through Iowa and then southerly through Missouri to join the Missouri River after flowing through a four-mile cutoff. Beginning in 1949, this flood control cutoff diverted the Chariton River directly into the Missouri River at a point approximately 12 miles upstream from its natural mouth. This cutoff separated the Chariton River from its tributary, the Little Chariton River, which is now an independent basin and tributary of the Missouri River.

The Chariton River is approximately 170 miles long and drains an area of 2390 square miles of which 925 square miles lie in Iowa and 1465 square miles lie in Missouri. The river slope is fairly uniform ranging from 1.5 to 2.5 feet per mile with an average of approximately 2 feet per mile. The river has been substantially channelized. Originally the Chariton River meandered a distance of over 200 miles from the Missouri-Iowa border to its mouth. That distance is now approximately 95 miles.

The Chariton River basin is long and narrow and extends nearly due north from the mouth to the Missouri-Iowa state line. The maximum length of the basin is about 140 miles and the maximum width is about 25 miles. The topography of the Chariton river basin is typical of the area consisting of rolling or gently undulating glacial plains divided by deeply eroded valleys. The only major impoundment in the basin is Rathbun Lake which is a Corps reservoir located in the upper basin in Iowa, approximately 140 river miles above the mouth of the Chariton River. (Reference Nos. 7 & 10).

9. Little Chariton River - RM 227.3

The Little Chariton River is a left bank tributary of the Missouri River which drains an area of approximately 691 square miles in north-central Missouri. The Little Chariton River was originally a part of the Chariton River basin. In 1949 a flood control cutoff on the Chariton River was constructed which started above the mouth of the Little Chariton and diverted the Chariton directly into the Missouri River. This made the Little Chariton River an independent basin draining directly into the Missouri River. The Little Chariton River is formed by the confluence of Middle Fork and East Fork at a point 17 miles above its mouth, and it flows into the Missouri River through the old, natural Chariton River channel.

The Little Chariton River basin is approximately 60 miles long with a maximum width of approximately 20 miles. The basin lies to the east of the Chariton River basin, and is entirely within the state of Missouri. The basin has a rolling topography with surface deposits of glacial till and loess overlying bedrock of Pennsylvanian Age. The Little Chariton River basin has two major impoundments -- Thomas Hill Reservoir on Middle Fork which is privately owned and Long Branch Lake on East Fork which is a Corps reservoir. (Reference Nos. 3 & 7).

10. Blackwater/Lamine River - RM 202.5

The Lamine River, with its major tributary, the Blackwater River, is a right bank tributary of the Missouri River, draining an area of 2640 square miles in west-central Missouri. The Lamine River, flowing in a northerly direction, is joined about ten miles upstream from its mouth by the Blackwater River, flowing in an easterly direction. The mouth of the Lamine is about five miles upstream from Boonville, Missouri. There are no major impoundments in the Blackwater/Lamine River basin.

The Lamine River originates in the southeastern part of the joint basin at the confluence of Flat Creek and Richland Creek, and meanders 64 river miles before reaching the Missouri River. Together with Flat Creek, its length is about 102 miles. Exclusive of the Blackwater basin, the Lamine River drains an area of 1090 square miles. The Lamine River channel slopes vary from about seven feet per mile in the upper basin to about two feet per mile along the lower main stem. The steeper channel slopes of the Lamine River permit faster discharge of flood flows than on the Blackwater River. The pronounced topographic relief contributes to quick runoff, resulting in frequent severe flood peaks of relatively short duration.

The Blackwater River and its tributaries drain 1550 square miles of the north and west part of the joint basin. The Blackwater River is 104 miles long and its chief tributaries are Salt Fork and Davis Creek. The Blackwater River channel slopes vary from about five feet per mile in the upper basin to about one foot per mile in the central and lower reaches. Topographic relief is not pronounced and runoff occurs at a comparatively moderate rate. (Reference no. 22).

11. Osage River - RM 130.2

The Osage River is a right bank tributary of the Missouri River which rises in east-central Kansas and flows eastward through west-central Missouri to join the Missouri River near Jefferson City, Missouri. The upper Osage River which is in Kansas and Missouri is called the Marais des Cygnes River. That portion of the river which is named the Osage River is entirely in Missouri and is formed by the confluence of the Little Osage and Marais des Cygnes Rivers. The Osage River is 262 miles long and the Marais des Cygnes River is 254 miles long. These two rivers combine for a total length of 516 miles and drain an area of 15,300 square miles, of which 28% is in Kansas and 72% is in Missouri.

The Osage-Marais des Cygnes River basin is approximately 250 miles long from west to east and has a maximum width of 100 miles. Headwater elevations reach 1450 feet N.V.G.D. and valley lands near the mouth of the Osage River lie at 520 feet N.G.V.D. River slopes of the Marais des Cygnes River average more that two feet per mile. River slopes of the Osage River average approximately 1.4 feet per mile.

The western part of the basin is in the Osage Plains area and is characterized by gently rolling uplands. The eastern part of the basin enters the Ozark Highlands Region and is rugged and hilly with deep, narrow valleys. Major impoundments in the Osage-Marais des Cygnes River basin are the Lake of the Ozarks, which is a hydroelectric power project of the Union Electric Company of Missouri and has a normal power-pool area of 60,000 acres, and six Corps reservoirs including Harry S. Truman Reservoir which has a full flood-control pool area of 209,300 acres. (Reference Nos. 6 & 15).

12. Gasconade River - RM 104.5

The Gasconade River is a right bank tributary of the Missouri River that rises in south-central Missouri and follows a northeasterly course. The river is about 265 miles long and drains an area of 3600 square

miles south of the Missouri River. The channel slope ranges from 0.8 to 6.2 feet per mile, with an average of 2.5 feet per mile. The average discharge volume over the period of record is 2490 cfs. The drainage area is mostly Ozark Plateau region with steep hillsides and ridges covered with timber. The Gasconade River is extremely meandering in character and flows through small alluvial valleys. The basin has many springs which contribute to stream flow. There are no major impoundments in the basin. (Reference No. 11).

III. THE UNET MODEL

The Kansas City-Missouri River UNET model was developed by the Kansas City District and Dr. Robert Barkau to represent the unique character of the Missouri River. The data necessary to develop the UNET model was assembled, including the Missouri River and tributary geometry, the levee data, and the observed discharge and stage data. Tributary routing reaches were added to the mainstem reaches and the model was run between USGS gauges. The mainstem reaches with tributaries were assembled into one continuous UNET model of the lower Missouri River. After calibration of this model to 1993 flood conditions, the alternative simulations were analyzed.

A. River Geometry

The geometry of the river system is described by cross-sections that extend from bluff to bluff. The Missouri River cross-sections used were compiled in the 1970's for the Missouri River Restudy (Reference No. 21) which utilized the Kansas City Backwater Program. The overbank geometry of the cross-sections was taken from two-foot contour maps which were developed by photogrammetry. The channel sections were developed from hydrographic surveys from the 1970's. The cross-sections were translated from Kansas City Backwater format into HEC-2 format, and then into UNET format. Some of the cross-sections did not extend beyond the federal levee systems. Overbank geometry for these cross-sections was augmented from USGS 7.5 minute series quadrangle maps. The distance between cross-sections averages from 0.5 mile to 1.0 mile. Total number of cross-sections on the mainstem exceeds 800. All tributary cross-sections were developed from USGS 7.5 minute series quadrangle maps. The distance between cross-sections on tributaries varies from approximately 5.0 miles to 0.5 miles.

B. Boundary Conditions

The discharge hydrograph from a Missouri River USGS gauging station is the upstream boundary condition inflow hydrograph for the beginning reach of the Kansas City-Missouri River UNET model. There is a USGS gauging station at the Kansas City district boundary at Rulo, Nebraska, but during large flood events, significant overland flow occurs which bypasses the gauge. For the 1993 flood event, approximately 30% to 40% of the peak flow bypassed the Rulo gauging station. Therefore, data from the Rulo gauge, for events the magnitude of the 1993 flood, are not suitable as an upstream or downstream boundary condition for UNET.

For calibration purposes, the discharge hydrograph at the next USGS gauge downstream from Rulo is used as the upstream boundary condition inflow hydrograph for the Kansas City model. For simulation of the alternatives, the discharge hydrograph as computed by the Omaha-Missouri River UNET model at this gauge, is used as the upstream boundary condition inflow hydrograph. Discharge data from USGS gauging stations are used as the upstream inflow hydrographs for all the tributaries modeled as routing reaches.

The downstream boundary condition for the UNET model is a stage hydrograph at a gauging station near the confluence of the Missouri and Mississippi Rivers. Ungauged inflow is modeled as uniform lateral inflow distributed along mainstem and tributary reaches as necessary. All observed discharge data are USGS daily flows. All observed stage data are Corps six-hour stage readings, except for two of the tributaries which have Corps daily stage readings.

C. Levees

For UNET, the Missouri River agricultural levees were modeled as systems. Where levee districts were contiguous, the levees were considered to be part of one levee system which encompassed one protected area. The minimum protected area of a levee system was established as a criteria to be 100 acres. Levee districts were aggregated into systems such that the protected area was greater than 100 acres. Levee properties, such as top of levee elevation, surface area encompassed by the levee, and upstream and downstream river mile, were determined for each individual levee system. These properties were used to build the UNET levee parameter files, called "include" files, which the program refers to during a UNET run.

During the 1993 flood, the Missouri River experienced three flood crests. Many of the agricultural levees within the Kansas City District failed as flood stages exceeded the design height of the levees by several feet. On the third and highest crest, virtually all agricultural levees were overtopped and there was significant overbank flow. The UNET version in use at the beginning of the FPMA, simulates levee systems as storage cells defined by surface area and height of levee above the ground elevation. In the model levees are failed based on a time of failure or based on river stage versus top of levee elevation. This methodology was sufficient for modeling the first two crests on the Missouri River during the 1993 flood, but it was inadequate for modeling the third crest. To overcome this problem, a unique levee algorithm was developed and programmed for Kansas City's version of UNET. This new UNET levee algorithm simulates the unique mode of failure of the Missouri River levees that occurred during the 1993 flood event.

The Missouri River levees failed early in the 1993 flood event, and subsequently the protected area behind the levees filled with water from the river. During the final third crest, the levees degraded and the floodplain behind the levees actively conveyed flow. The Missouri River functioned under two geometric conditions: one in which levees constrained the flow to the channel, but provided storage behind the levees; and the second in which the levees no longer constrained the flow, and the overbank actively conveyed flow as if the levees did not exist (Reference No. 1).

With the new levee algorithm, levees are modeled in UNET in the following manner: When a levee fails at a breach and subsequently fills, the flow through the breach section depends on the elevation of the river and the elevation of the water in storage behind the levee. The water surface inside the levee interior is assumed to be horizontal. When the river discharge exceeds a specified flow, or when the river elevation exceeds a specified elevation, then the levee storage cross-sectional area and conveyance are added to the river cross-sections and the program routes flow through the channel and the entire width of the floodplain. When the flow falls below a specified discharge, or the river falls below a specified elevation, the levee storage cross-sectional area and conveyance are subtracted from the cross-sections and the river once again interacts with the levee through the breach.

The point at which the model changes from storage routing to floodplain routing is specified by flow. UNET uses elevation only if a flow is not specified. These flow values can be determined from rating

curves observed at the gauges. Plate KC-2 is a rating curve at the Boonville gauge. In this figure, the flow dramatically increases with stage beginning at approximately 300,000 cfs, which is the point where the floodplain actively begins to convey flow.

D. Calibration

The Kansas City-Missouri River UNET model was calibrated to reproduce observed stages at river gauges for the 1986 and the 1993 flood events. The primary factor for adjusting the model is Manning's "n" which is an estimate of the friction force of the boundaries on the flowing water. For large rivers, Manning's "n" varies with depth, because the relative size of the roughness elements, such as the dunes in the river bed, the height of vegetation, etc., declines with increasing depth. The object of the calibration process is to determine the variation of Manning's "n" with depth. The effect of the friction force from downstream is shown in the rating curve (stage versus flow) at a gauging station; hence, the variation of Manning's "n" with depth is represented in the relationship. The automatic calibration option of the UNET program adjusts conveyance at the cross-sections between gauging stations such that the model reproduces the stages at the upstream rating curve under steady flow.

The Missouri River rating curves were developed at the principal gauging stations from observed stage data and computed flow data for the years 1973, 1986, and 1993. The model was calibrated to reproduce these rating curves. Since, this study concentrates on the 1993 event, the model's existing conditions calibration was adjusted to specifically reproduce the 1993 stages and flows.

E. Systemic UNET Modeling

Analysis of the alternatives was performed on a system-wide basis encompassing the whole of the Missouri/Mississippi Basin study area. In order to conduct systemic analyses, it is necessary to transfer data between UNET models. The locations at which data would be transferred were selected based on availability of dependable gauge data at the selected location, Corps district boundaries, backwater conditions, and cross-section geometry.

The Kansas City-Missouri River UNET model is sandwiched between the Omaha-Missouri River UNET model and the St. Louis-Mississippi River UNET model. The Omaha model supplied the upstream flow hydrograph to begin the Kansas City model. In turn, the Kansas City model supplied the upstream flow hydrograph from the Missouri River to the St. Louis model. A UNET model geometry overlap reach is needed to accurately define the upstream and downstream boundary conditions at each transfer location.

Upstream and downstream of the districts' boundary at Rulo, data was exchanged between the Kansas City and Omaha districts. As a matter of joint District agreement, due to the previously discussed difficulties of using the Rulo gauge data as a boundary condition and due to the relative lengths of the two models, hydrograph information was transferred between the Omaha and Kansas City models at the St. Joseph, Missouri, gauging station. Omaha District modeled from Omaha, Nebraska (RM 615.97) to downstream of St. Joseph at RM 410.0. Kansas City District modeled from the St. Joseph gauge to the St. Charles, Missouri gauge (RM 28.2).

Data was also exchanged between the Kansas City and St. Louis districts on the lower Missouri River from Hermann, Missouri, to the mouth at the confluence with the Mississippi River. The St. Louis model included the Missouri River as a tributary routing reach which needed to start at the farthest downstream rated gauge, which is at Hermann, Missouri. Therefore, it was necessary for the St. Louis

District to model the Missouri River reach of the St. Louis model from the Hermann gauge to the confluence with the Mississippi River. Hydrograph information was transferred between the Kansas City and St. Louis models at the Hermann gauging station (RM 97.9).

IV. SIMULATION OF ALTERNATIVES

In order to address the questions being studied for the FPMA, systemic hydraulic analyses were undertaken of several alternatives. Systemic alternative analysis necessitates transfer of data between UNET models. The upstream model provides inflow data for the downstream model, which utilizes the data and then produces inflow data for the next downstream model. This sequential operation of passing and processing of data was termed the pass-off. Three pass-off meetings were convened of the districts involved with UNET modeling for the FPMA. Dr. Barkau also attended these meetings. Between each meeting, the Kansas City model and the new levee algorithm were adjusted and refined. The Kansas City model's integrity improved with each refinement as evidenced by increasingly more accurate reproduction of the observed hydrographs.

Results of the systemic modeling effort were verified by means of an iteration process. After the first run of the UNET model, the downstream model passed back a stage hydrograph to the upstream model to be used as the upstream model's downstream boundary condition for a second run of the model. After the second run was performed, the flow hydrographs produced by the first and second runs of the upstream model were compared at the transfer location. Error limits of less than one percent of peak discharge had been established as a criteria. For all alternatives which were iterated, results were within one percent of peak discharge.

A. Description of Alternatives

Alternatives were modeled which simulated various changes to agricultural levees on the mainstem of the Missouri River. Agricultural levees are defined as all those levees not specifically designated as urban levees for the purposes of this study. The urban levees within the limits of the Kansas City model are L-455 and R-471-460 in the St. Joseph area; Fairfax-Jersey Creek, North Kansas City, C.I.D., East Bottoms, and Birmingham in the Kansas City area; and Monarch-Chesterfield, Riverport, and Earth City in the metropolitan St. Louis area. Actually, the R-471-460 levee unit is a federally designed agricultural levee, but for this study it was considered to be an urban levee.

Other alternatives simulated existing 1993 conditions or changes to existing 1993 flow conditions. The following is a description of each systemic alternative and the results produced by the Kansas City-Missouri River UNET model. For summarizations of the results, see Tables KC-1 through KC-5. The stage hydrographs produced by UNET for each gauge on the Missouri River mainstem are plotted on Plates KC-3-A through KC-7-E.

1. Alternative No. 1: Base Conditions

The first alternative analyzed was the existing 1993 flood conditions, also termed base conditions. The calibrated UNET model was used to perform the simulation with USGS inflow hydrograph data. Stage and flow hydrographs computed by the model were compared to observed stage and flow hydrographs at the USGS rated gauges on the mainstem of the Missouri River. The location of these gauges are St. Joseph, Kansas City, Waverly, Boonville, and Hermann, Missouri. Differences between computed and

observed stages at these locations ranged from 0.1 to 0.3 feet. These results are summarized in Table KC-1 and on the stage hydrographs, Plates KC-3-A, 4-A, 5-A, 6-A, and 7-A.

2. Alternatives No. 2 and No.3: Remove Agricultural Levees

These alternatives analyzed the impacts of removing all agricultural levees along the Missouri River. Urban levees remained intact. For Alternative No. 2, the overbanks were modeled to simulate an agricultural regime in the floodplain. For Alternative No. 3, the overbanks were modeled to simulate natural ecological succession in the floodplain. Only the roughness value, or Manning's "n" value, was adjusted to reflect different forms of land use within the overbank. Other factors affecting conveyance were not evaluated in detail.

Removing all agricultural levees provided significant additional flow area since cross sections are as much as several miles wide. In actuality, natural and constructed obstructions within the conveyance area will restrict effective flow width to a value much less than the cross section width. However, the UNET model uses the entire cross section width, and therefore overstates the available flow area. Modifying the UNET model to accurately reflect the actual conveyance changes at every cross section was not possible for this assessment. Therefore effective flow width, and other factors which reduce cross section conveyance, were accounted for in the UNET model by adjusting roughness values.

Roughness values were selected to provide a reasonable lower and upper bound for computed results. Various forms of land use within the overbank area will have considerably different roughness values. Generally accepted Manning's "n" values are 0.04 for agricultural land use and 0.16 for a natural wooded floodplain. These roughness values were doubled to approximate a fifty percent reduction in the overbank effective flow area. Therefore, a roughness value of 0.08 was used to model agricultural activity and vegetation growth in the overbanks, and a roughness value of 0.32 was used to model natural vegetation growth in the overbanks. These values were used uniformly throughout the entire study area of the Missouri/Mississippi basins.

The roughness value adjustment did not reduce the area available for overbank flood storage. Therefore, the computed results for the levee removal alternatives should be regarded as estimates because of the assumptions discussed in the previous paragraphs. More accurate simulation of levee removals would require the construction of an entirely new model, and detailed studies to determine the long-term effects on conveyance of vegetation and sedimentation within the floodplain.

In all cases except one, agricultural vegetation in the overbanks produced lower peak stages at the gauges than natural vegetation. The exception occurred at the Kansas City gauge. In this instance, the gauge is located in an urban area with urban levees on each side of the channel and no overbank areas. When the computations of the model reached the beginning of the urban levee area upstream of the Kansas City gauge, all flow in the overbank area was transferred to the channel. The agricultural vegetation alternative had a greater discharge than the natural vegetation alternative through the Kansas City reach at the peak stage, while both alternatives had essentially the same velocities throughout the reach. This resulted in a greater peak stage for the agricultural vegetation alternative at the Kansas City gauge.

At the Boonville gauge and downstream, the natural vegetation alternative resulted in peak stages higher than the base conditions peak stages. This also occurred once for the agricultural vegetation alternative at the Hermann gauge. These results were the consequence of a combination of factors including the

narrowing of the floodplain below Glasgow, Missouri; the similarity of this alternative to 1993 flood flow conditions; and the change in timing caused by the removal of all agricultural levees.

The floodplain between Waverly and Glasgow has a maximum width of approximately thirteen miles. But in the vicinity of Glasgow, which is only thirty miles upstream from the Boonville gauge, the floodplain constricts abruptly. From Glasgow downstream, the floodplain is much narrower than upstream of Glasgow, averaging approximately two miles in width. The floodplain was immediately available to flood flows, since there were no levees to constrict flow until overtopping. This can exacerbate flood stages if timing is changed such that tributary and mainstem peak stages coincide.

At the times of the peak stages on the lower Missouri River during the 1993 Flood, the overbank functioned as if the levees did not exist. Therefore, on the lower Missouri River, the 1993 Flood was essentially a "no-levee" scenario. With higher "n" values in the overbanks for the natural vegetation alternative, velocities decreased and stages increased above the base conditions.

3. Alternative No. 4: Confine 1993 Flood Flows

For this alternative, all levees were raised infinitely high to completely contain the 1993 flood flows within the existing flow way between the levees. Levee locations and roughness values were not altered for this alternative. For all gauges, this resulted in an increase of the peak stage above base conditions peak stage.

4. Alternative No. 5: 25-Year Levees

The height of all agricultural levees was set to correspond with the Missouri River 25-year frequency water surface profile. The 25-year profile was obtained from the Missouri River Restudy (Reference No. 21). Historically, Missouri River levees usually fail at the upstream end first. Therefore, the levee failure mode used for this alternative was to fail the levee at the upstream end at the 25-year water surface elevation, and then have the levee overtop at the downstream end to simulate flow into and out of the levee system. For all gauges, this resulted in a decrease of the peak stage as compared to base conditions peak stage.

5. Alternatives No. 6 and No. 7: Runoff Reduction

These alternatives were intended to simulate reductions in the 1993 flood runoff volumes which might have occurred if there had been more upland retention storage available at the time of the flood. The purpose was to investigate the sensitivity of 1993 peak stages to reductions in runoff. For these alternatives, the observed 1993 flood inflow hydrographs from all gauged tributaries of the Missouri River were reduced by five and ten percent. This was accomplished by multiplying each ordinate of all tributary inflow hydrographs by the reduction factor, resulting in a total volume reduction. For the purposes of modeling, all inflow hydrographs were reduced by an equal percentage. But, in reality, runoff reduction would not be distributed equally over the total inflow hydrograph, nor would each tributary basin store the same additional percentage of its total runoff volume.

For all gauges, the five percent reduction in runoff volume resulted in a decrease of the peak stage compared to 1993 base conditions peak stage. In all cases, this decrease in stage was less than one foot, except at the Kansas City gauge which had a 1.1 foot stage reduction. For the ten percent reduction in runoff volume, the change in stage compared to 1993 computed stages varied from a maximum decrease

of 2.2 feet to a maximum increase of 1.2 feet. This variation in stage differences is the result of change in the timing of discharge due to reduced runoff volumes and thus, a change in the time of levee failures and of the switch from storage cells to overbank routing.

6. Alternative No. 8: No Federal Reservoirs

Simulation of this alternative was performed to assess the impact of federal reservoirs on 1993 flood flows. The effect of reservoirs on Missouri River stages was determined by using the natural inflow hydrographs reconstituted for the 1993 flood without reservoir holdouts. These hydrographs were reconstituted for tributaries which have federal reservoirs in their basins. The Water Control Section of the Kansas City District used their Benefits Program to produce the reconstituted hydrographs. The Benefits Program routes the reservoir holdouts from the reservoir downstream to damage points or gauges. The program calculates stage reduction at that location and the fair-share distribution of the reduction among the projects. This program was run for all Missouri River tributaries in the District which have federal reservoirs. Federal reservoirs include all those under the jurisdiction of the Corps of Engineers and the Bureau of Reclamation.

The result of this simulation was an increase in the peak stage at all gauges compared to the base conditions peak stage. The increase in peak stage was greater than 3.5 feet at more than half of the gauges. The minimum increase in the peak stage was 0.4 feet at the St. Joseph gauge. The maximum increase in peak stage was 5.1 feet at the Kansas City gauge, which is just downstream of the confluence of the Kansas and Missouri Rivers. The Kansas River Basin also experienced record flooding during July and August, 1993. There are eleven Bureau reservoirs and seven Corps reservoirs in the Kansas River Basin.

7. Alternative No. 9: Levee Setbacks

This alternative examined the effect of levee setbacks on flow conditions and flood stages. Setback of a levee refers to moving the levee from its present location to a new location that is further from the river. Levee setbacks are intended to increase the cross section flow width and reduce the constriction of the flow area that occurs in a narrow channel. From the results, it can be concluded that for the Kansas City model the increase in available flow area was ineffective and apparently served only as a temporary storage area.

All agricultural levees were set back from their existing location such that the width of the floodway was increased by 50%, and a minimum floodway width of 5000 feet was maintained. The floodway is defined as the area between the left and right encroachment stations of a cross section. Encroachments are located at the stationing of the levee crown on the left and right banks, or at the natural limits of a cross section, such as a bluff line. When the levees were set back, some levee systems fell below the minimum protected area criteria and these systems were eliminated from the analysis. The existing top of levee elevations were maintained. Urban levees were not set back.

The results of this alternative were similar to 1993 base conditions. Peak stage differences from base conditions peak stages ranged from -0.5 feet at Kansas City to +1.0 foot at Boonville. Maximum discharge change from base conditions was +3.9% at St. Joseph and the minimum discharge change was +0.3% at Hermann. Most all of the agricultural levees failed, except some of those above St. Joseph, Missouri.

V. CASE STUDIES AND SPECIAL STUDIES

A. Setback Study Reach

The UNET model was used to analyze the effect of an isolated levee setback on flow conditions throughout the FPMA study reach in the Kansas City District. The effect of levee setbacks on flow conditions and flood stages was examined. Setback of a levee refers to moving the levee from its present location to a new location which is further from the river. Levee setbacks are intended to increase the cross section flow width and reduce the constriction of the flow area that occurs in a narrow channel. However, the flow area increase may be offset by an elevated roughness condition.

The setting back of levees would affect roughness values within the cross section. Roughness for the area between the existing levee and the setback levee locations might change due to changes in land use. Estimating the changes that would transpire due to vegetative growth, sediment deposition, and other developments within the expanded floodway, would be speculative and was not investigated. The probable combinations of land use changes, geometry changes, and roughness changes were not examined for their possible effects on the results computed with the UNET model.

All levees within the study reach were set back from their existing location such that the width of the floodway was increased by 50%, and a minimum floodway width of 5000 feet was maintained. The floodway is defined as the area between the left and right encroachment stations of a cross section. Encroachments are located at the stationing of the levee crown on the left and right banks, or at the natural limits of a cross section, such as a bluff line. When the levees were set back, some levee systems fell below the minimum protected area criteria and these systems were eliminated from the analysis. The existing top of levee elevations were maintained.

The isolated setback reach location selected for this case study was the reach of the Missouri River from approximately river mile 486 where Little Tarkio Ditch flows into the Missouri River, to the downstream end of levee L-476 at approximately river mile 454. All levee systems from river mile 486 downstream to river mile 454 were setback. Within this reach during the 1993 flood, the levee systems of Windle, R-500, L-488, and R-482 were overtopped.

When the levees were set back and 1993 flood flows were routed through this study reach, only the Windle levee system was overtopped. The setback of levees within this reach had little effect on peak stages downstream of the reach. The peak stage at the St. Joseph, Missouri, gauge was reduced by only 0.04 feet. Peak stages at the Kansas City, Missouri, gauge and downstream were not effected. The following table (Table KC-6) lists the levees that were set back and their performance. Levee parameters are listed in Table KC-4.

TABLE KC-6
MISSOURI RIVER
LEVEE PERFORMANCE
SETBACK STUDY REACH

LEVEE SYSTEM NAME	SETBACK STUDY REACH PERFORMANCE
Windle	overtopped
R-500	not overtopped
L-497	not overtopped
L-488	not overtopped
R-482	not overtopped
L-476	not overtopped

B. Monarch-Chesterfield Levee Case Study

The Monarch-Chesterfield Levee case study was a cooperative effort of the Kansas City District and the St. Louis District. The Monarch-Chesterfield levee extends from approximately river mile 46.0 downstream to river mile 38.5. This levee breached during the 1993 flood. The local community has requested that the Corps study increasing the level of protection of the levee to a 0.2% chance (500-year) flood. For this case study, the Monarch-Chesterfield levee was analyzed to obtain the results of increasing the levee's level of protection to withstand the 1993 flood. The UNET model was used to calculate the flood elevation impacts of the higher levee upstream and downstream of the Monarch-Chesterfield levee area.

When the Monarch-Chesterfield levee was raised so that it would not fail for 1993 flood conditions, the maximum water surface elevation increase was 0.8 feet just upstream of the Monarch-Chesterfield levee. Riverport levee (river mile 30.0 to 29.6) and Earth City levee (river mile 29.5 to 27.1) are urban levees located downstream of the Monarch-Chesterfield levee. They did not fail during the 1993 flood and they did not overtop when the Monarch-Chesterfield levee was raised above 1993 flood levels.

Additional information on this case study is contained in paragraph 12.a., page SL-11, of the St. Louis District's hydraulics and hydrology report in Appendix A of the Floodplain Management Assessment (FPMA) Final Report.

C. Bridge Constrictions Evaluation:
BRIDGES, THEIR OPENINGS, AND THE ROADWAY EMBANKMENT LEADING TO THE ABUTMENTS

Most bridges spanning a river in the United States are designed with an opening sufficient to pass a flood discharge of an identified magnitude. Some bridge openings were sized by empirical methods which reflect the local geographical conditions or by another methodology peculiarly dependent on the designer's

experience. Older bridges reflect the technology as well as the economic conditions of the era in which they were built. Newer bridge designs recognize Federal standards to control encroachments into a floodway. More consistent and generic criteria have been adopted which have caused an increase in the amount of bridge opening that must be provided to pass a given, analytically derived flood event (either synthetic or reduced from existing data) without generating a calculated measurable adverse impact on the upstream community or surroundings.

Under current procedures, bridge openings are usually sized and checked using a step-backwater analytical method which yields a worse scenario for energy losses than the bridge may actually generate. The assumptions of this method are basic to a uniform steady state condition and are equivalent to assuming a constant supply of water without changes in magnitude, either with time or space. Under most design conditions, the user of a step-backwater analysis does not have to properly distribute the discharge between the channel and overbank, but assumes that all the water is passing through and under the bridge. Further, the hydraulic designer applies sufficiently high expansion and contraction coefficients, assumes debris is piled against the piers blocking the opening, and uses rigid geometry boundaries which assumes that the bed does not scour or degrade. This type analysis yields higher energy losses than will ever occur in an actual river channel crossing. It may be readily observed that a step backwater analysis is not a good measure of bridge hydraulics, but does provide a good evaluation of a hydraulic constriction.

To develop what actually happens when a flood wave is moving through a river valley, consider the sketch shown on Plate KC-8-A. In this example, a river channel is located near the middle of the valley's flood plain. Figure A represents a flood hydrograph of a given magnitude. Figure B is a sketch in plain view and Figure C is a profile of the channel flow line, top of bank or elevation of the flood plain. Also indicated on Figure C is the water surface profile developed as the assumed flood wave shown in Figure A moves down the valley. The line above the water surface profile is the energy grade line. When water flows, the depth and the velocity represent two types of energy associated with the movement and they may be considered separately. The depth reflects the water surface profile, or potential energy. The velocity represents the kinetic energy. The energy grade line is a technique for representing the combined potential and kinetic energy. The energy grade line sums the local datum (the ground elevation at the bottom of the channel), plus the potential energy or depth of flow, plus the kinetic energy or velocity head in feet. The velocity is converted to feet by squaring the velocity and dividing by twice the acceleration of gravity.

For the flood wave hydrograph moving through the valley, a flood profile can be drawn depicting the water surface and an energy gradient. Generally, the energy gradient is only apparent along the bluff line or at a location where the velocity is forced to zero. Due to phenomena such as friction, eddies, or boils, energy is expended as water moves down a valley or a channel. For bridge constriction analysis, these types of losses are considered negligible.

As the flood depth exceeds the confinement of the channel, water flows into the flood plains. The total discharge is thereafter distributed in and across the whole valley until the excess runoff from the contributing watershed ceases. The flood wave then recedes and returns to within the channel once again. Figure D on Plate KC-8-A represents a cross section of the water surface across the flood plain and channel as measured from a weighted energy grade line.

The principle of the energy gradient can be applied to the question of bridge constrictions through the example of a bridge and a roadway embankment constructed across a valley. The bridge opening in the

example will pass the peak discharge represented by the flood hydrograph shown by Figure A on Plate KC-8-A. In this example, the bridge and roadway embankment are infinitely high so as not to be overtopped by the flood wave. Some fraction of the flood water will go into storage in the overbank as the flood wave moves downstream. The velocity in the flood plain will approach zero near the bridge and road embankment. As the velocity in the flood plain approaches zero, the depth increases to approach the weighted energy grade line. All the water will not pass under the bridge due to the distribution in the overbank.

As the volume of storage available in the overbank becomes occupied, the crest of the flood wave will be nearly equal to the energy grade line. The discharge under the bridge will then simultaneously approach the design discharge. On Plate KC-8-B, Figure B is a plan view, Figure C is the profile, and Figure D is the cross section representing the changes in water surface profile as distributed across the valley.

If a greater flood occurs than the bridge was designed to pass (greater than the flood described by Figure A, Plate KC-8-A), backwater effects representative of the constriction would be added to the conditions described above. As the total discharge increases with stage and time, so will the available storage in the flood plain. Again, as the crest approaches the bridge opening, the crest elevation will nearly equal the energy grade line because velocity in the overbank flow is forced to zero at the embankment. A discharge near the crest discharge will pass through the bridge opening. However, because the crest discharge exceeds the design of the opening, the energy grade line and the water surface will rise at the embankment.

This raised water surface profile will extend upstream until it intersects an equivalent water surface. By considering friction and other losses, this distance could be extended a little farther upstream depending on the slope of the water surface before a normal depth will be regained. Other factors that make each bridge unique in this sort of example are the involvement of girders or other obstructions besides the bridge piers, and the passage of flow over road embankments landward of the bridge abutments. See Plate KC-8-C, for profiles and other pertinent drawings representing this example.

All bridges, both railroad and highway, which presently span the Missouri River channel are theoretically sufficient to pass flood discharges equal to or greater than a 100-year flood with minor stage increases upstream. Unfortunately, this fails in practice whenever a flood event exceeds the upstream channel confinement and the flood discharges are not contained to the width of the bridge openings. That is, while a 100-year flood discharge might pass through a particular bridge opening, only part of an actual 100-year flood might pass beneath the bridge. When a portion of the flood volume goes into flood plain storage, and the roadway is on a low fill embankment, water often overflows or breaches the roadway and continues down the flood plain. If roadway fill is high, a measurable pile-up will occur at the bridge equal to the energy grade line.

Generally, the water surface downstream of the bridge and embankment will immediately return to the stage or depth normal to natural conditions. The water surface in the overbank will approach the energy grade line along the edges of the flood plain. This is true for all frequency of events that exceed flood stage.

During the 1993 flood, conditions similar to those described above were observed at the following locations:

* Prior to the breaching at the railroad crossing near Rulo, Nebraska
* I-635 above Kansas City, Missouri
* At the railroad crossing near Glasgow, Missouri
* I-70 near Rocheport, Missouri
* Highway 63 at Jefferson City, Missouri

Where the Federal levees near St. Joseph, Missouri, or through the metro area of Kansas City, Missouri, confined the 1993 food, little to no measurable losses were observed which were attributable to bridge or roadway embankments encroaching on the flood plains.

At St. Charles, Missouri, a considerable swell head was observed during the 1993 flood due to extremely high velocities in the channel with little bed scour. Supercritical conditions existed in the reach, with anti-dunes moving upstream and breaking on the surface. A large portion of this increase in upstream stage was due to the bed conditions being reflected on the surface.

As a result of the total flood plain devastation during the final third peak of the 1993 flood, the effects were very local and only differed because of the energy level of the discharge existing in the Missouri River channel. However, because the Missouri River discharge slope is steep, a backwater effect due to these types of encroachments usually dissipates very rapidly. The induced effect would equal the difference caused by the encroachment divided by the slope of the river. As an example, the slope of the 1993 flood for the last crest moving downstream averaged about 0.8 foot per mile. If an encroachment caused a 1.0 foot rise in stage, then approximately 1.25 miles would show some backwater prior to depths or stages returning to approximately normal.

If no levees along the Lower Missouri River failed, then probably a flood between the 25- and 10-year event would pass without any attention being given to the effects of bridges and/or their roadway embankments.

Among the numerous combinations of bridge openings and embankments crossing the Missouri River flood plain, each alignment is unique to the local topography and the sinuosity of the channel. Each configuration results in part from the economic climate at the time of construction. Because the slope of the Missouri River is fairly steep for a subcritical stream, the backwater effects from encroachment are not cumulative and dissipate very rapidly. Natural encroachments are one of the primary causes of increased flood stages and do accumulate effects upstream. However, in bridge analysis, these encroachments are considered to be a natural part of the river's environment.

Under present standards for a new bridge or embankment in the flood plain, especially within the designated floodway, each proposal is hydraulically analyzed and examined against the standards of several Federal agencies before it receives all the necessary permits for construction. The hydraulic examination uses the current physical conditions and assumes geotechnical aspects are not subject to failure. In contrast to these assumptions, the Missouri River's channel bed is constantly in motion and will scour during a flood to accommodate increases in stress, particularly from velocity. As the channel velocity increases with increasing discharge or stage, the bed will degrade to allow the channel to transport a greater portion of the discharge. Discharge measurements collected by the USGS show that prior to the private levees failing along the Missouri River, the channel capacity was accommodating some 90 to 95 percent of the total discharge at about 8.0 to 9.0 feet per second average velocity. This variable tendency of the Missouri River bed means that standard bridge analysis techniques are more

conservative when applied to Missouri River bridges than to streams where the channel bed is more resistant.

Most hydraulic analyses do not examine for exposed flank conditions, or for changes in a velocity momentum vector, but perform a hydraulic analysis based on FEMA guidelines. FEMA criteria do not allow for any measurable increases in water surface elevation within the designated floodway. A computed change of 0.01 foot is FEMA's guideline, which is much less than can be accurately measured in the field.

In summary, Missouri River bridges associated with high roadway embankments may have caused a backwater effect in the 1993 flood. Where they did occur, these effects were confined to a short distance immediately upstream of the bridge, were not systemic or cumulative along the river, and were generally attributable to unique local conditions. Modern methods for bridge opening design or sizing of a constriction tend to discount the potential for overbank flow losses. Consequently, whatever losses may be assumed in the design represent the worst case, are probably greater than losses that do exist, and yield some additional discharge higher than the bridge opening design.

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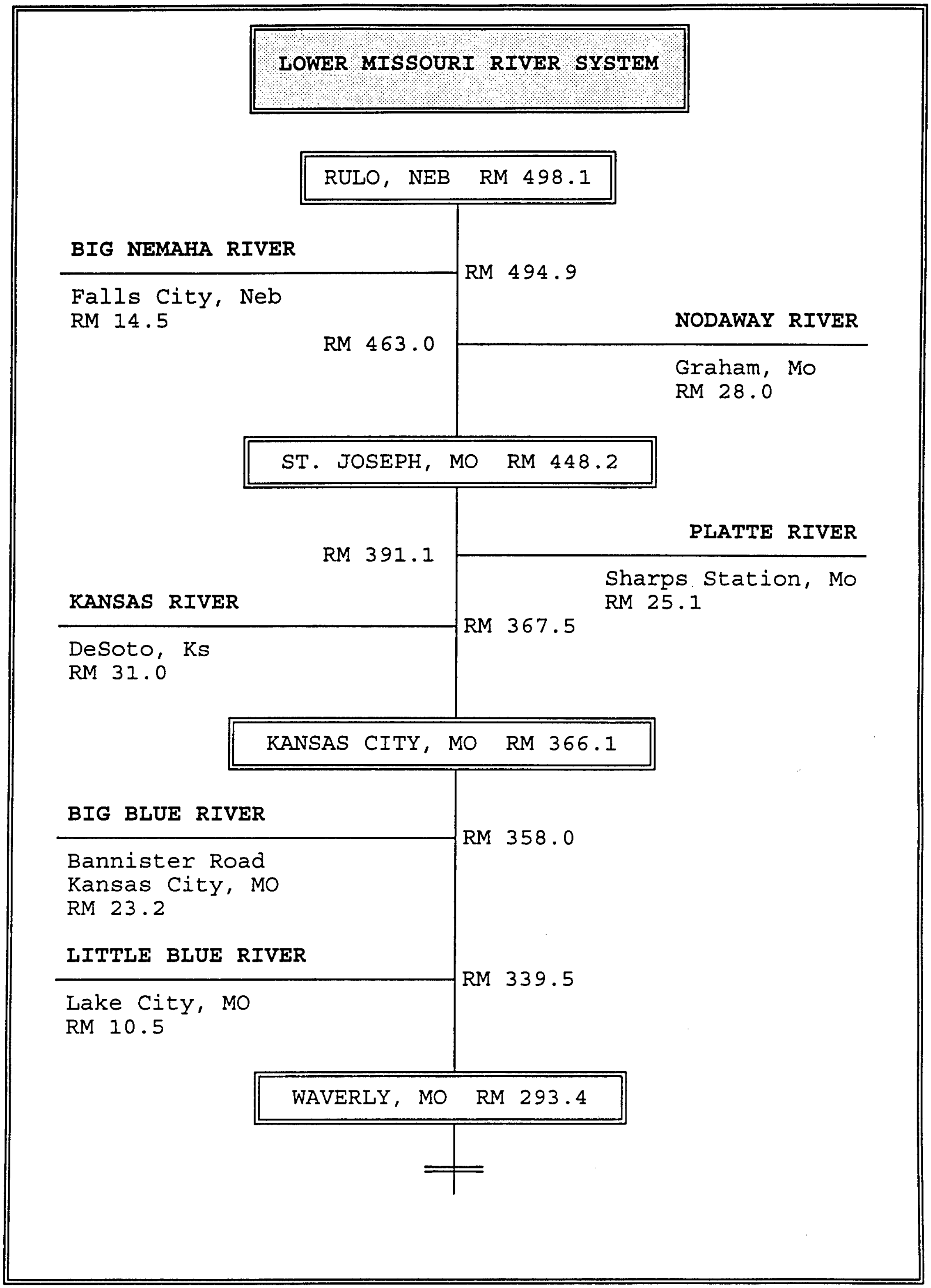

LOWER MISSOURI RIVER SYSTEM
RULO, NEB RM 498.1
BIG NEMAHA RIVER
RM 494.9
Falls City, Neb
RM 14.5
RM 463.0
NODAWAY RIVER
Graham, Mo
RM 28.0
ST. JOSEPH, MO RM 448.2
PLATTE RIVER
RM 391.1
Sharps Station, Mo
RM 25.1
KANSAS RIVER
RM 367.5
DeSoto, Ks
RM 31.0
KANSAS CITY, MO RM 366.1
BIG BLUE RIVER
RM 358.0
Bannister Road
Kansas City, MO
RM 23.2
LITTLE BLUE RIVER
RM 339.5
Lake City, MO
RM 10.5
WAVERLY, MO RM 293.4

PLATE KC-1

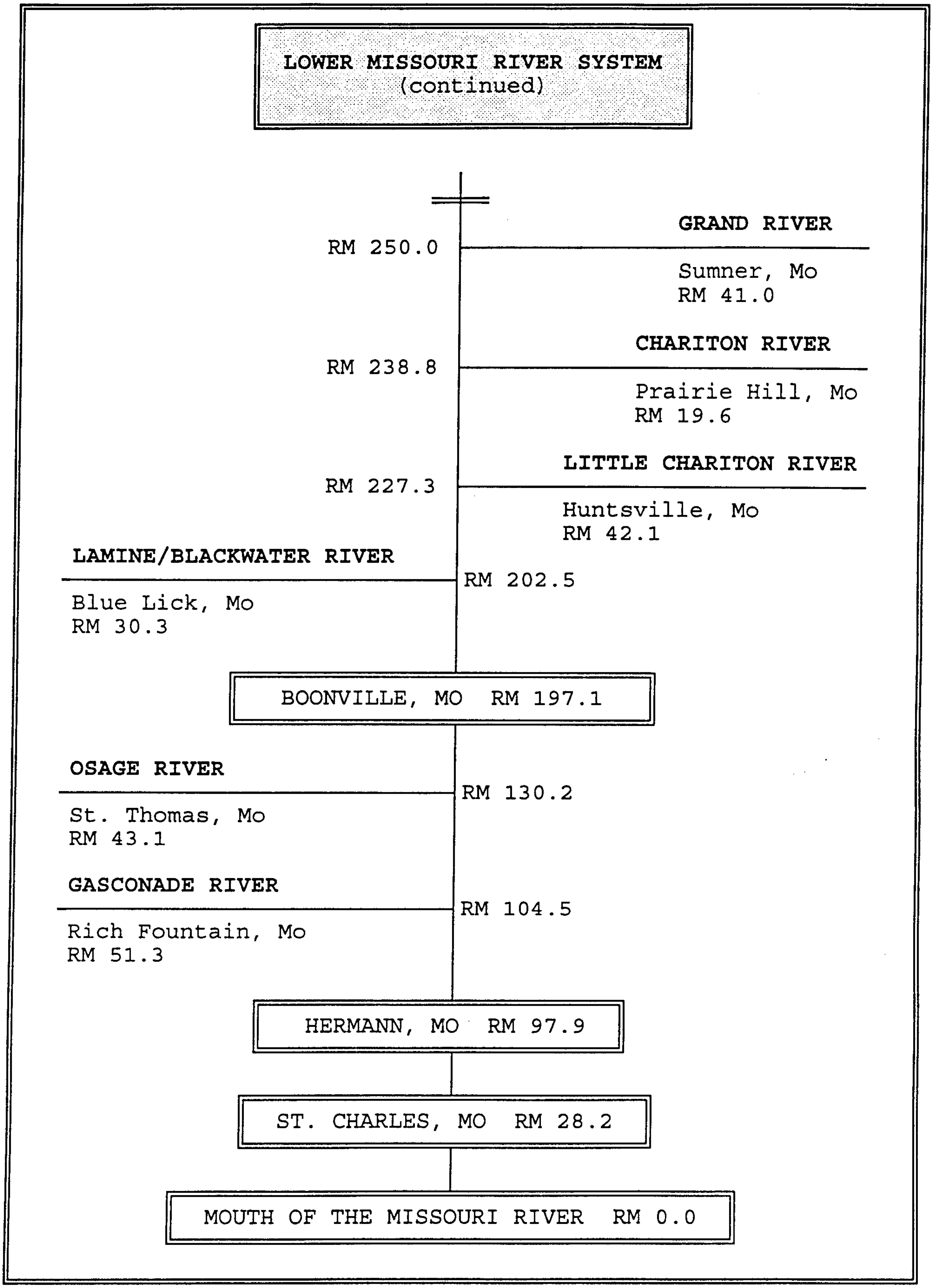
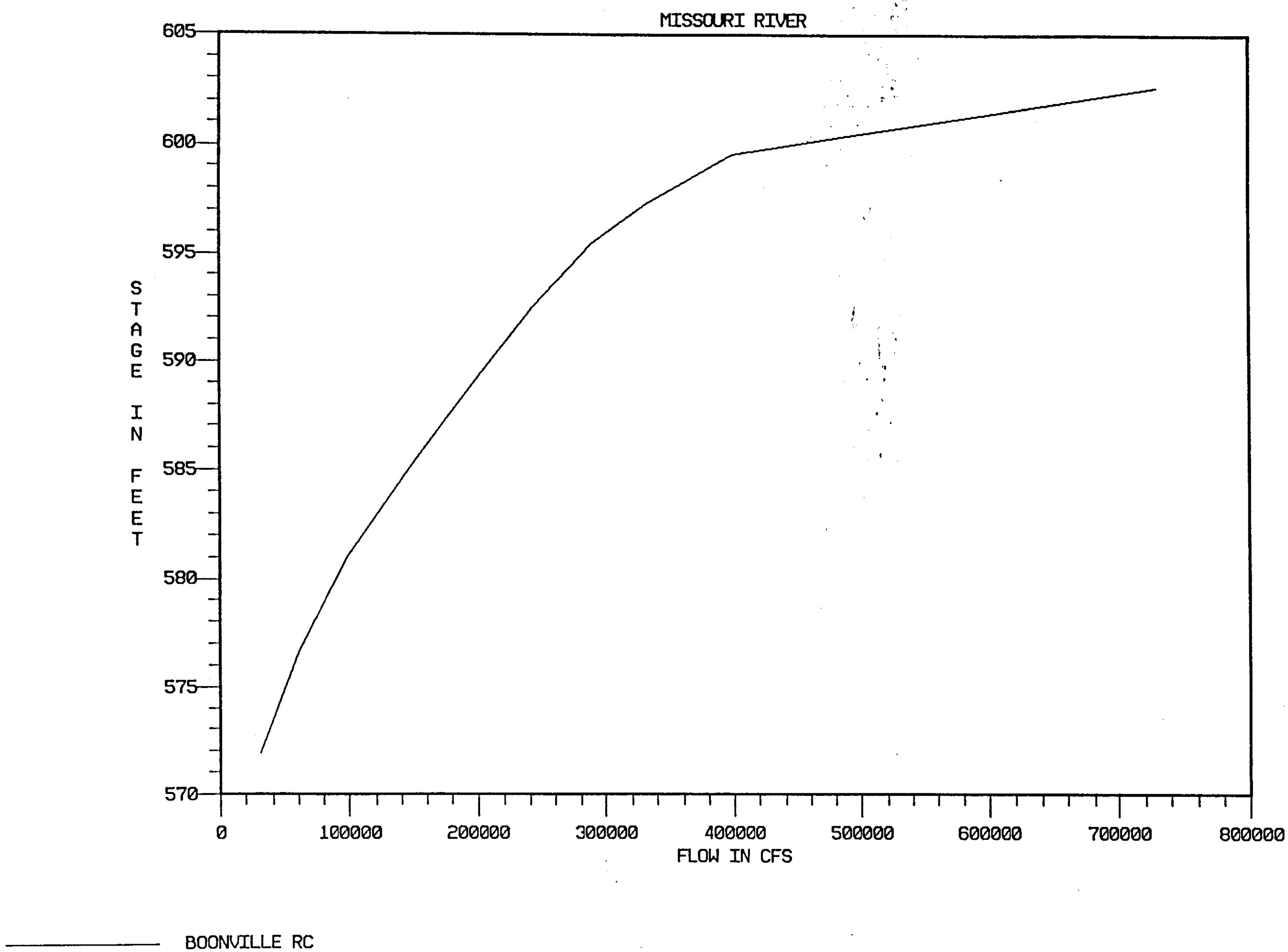
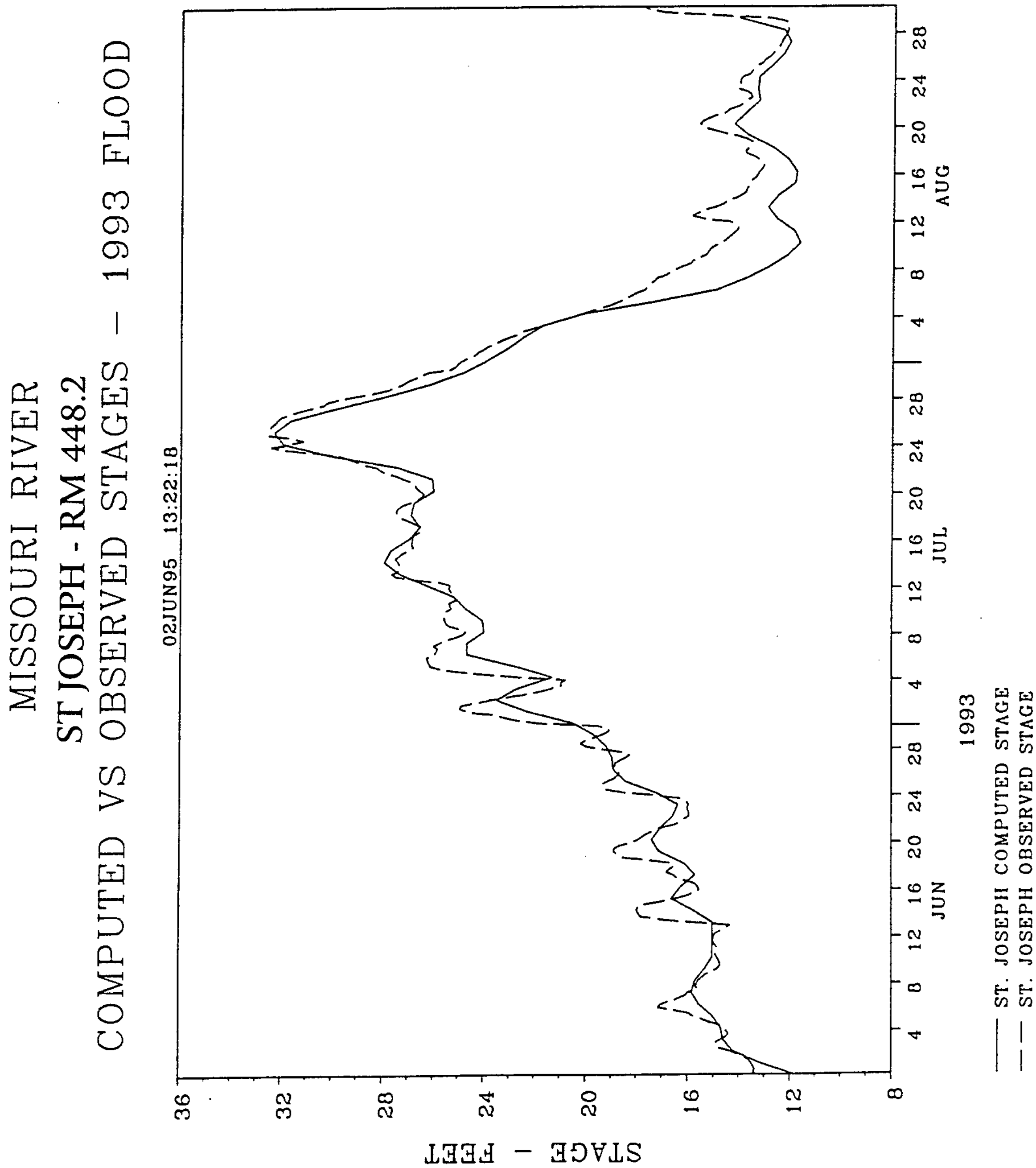

LOWER MISSOURI RIVER SYSTEM
(continued)
GRAND RIVER
RM 250.0
Sumner, Mo
RM 41.0
CHARITON RIVER
RM 238.8
Prairie Hill, Mo
RM 19.6
LITTLE CHARITON RIVER
RM 227.3
Huntsville, Mo
RM 42.1
LAMINE/BLACKWATER RIVER
RM 202.5
Blue Lick, Mo
RM 30.3
BOONVILLE, MO RM 197.1
OSAGE RIVER
RM 130.2
St. Thomas, Mo
RM 43.1
GASCONADE RIVER
RM 104.5
Rich Fountain, Mo
RM 51.3
HERMANN, MO RM 97.9
ST. CHARLES, MO RM 28.2
MOUTH OF THE MISSOURI RIVER RM 0.0

PLATE KC-1



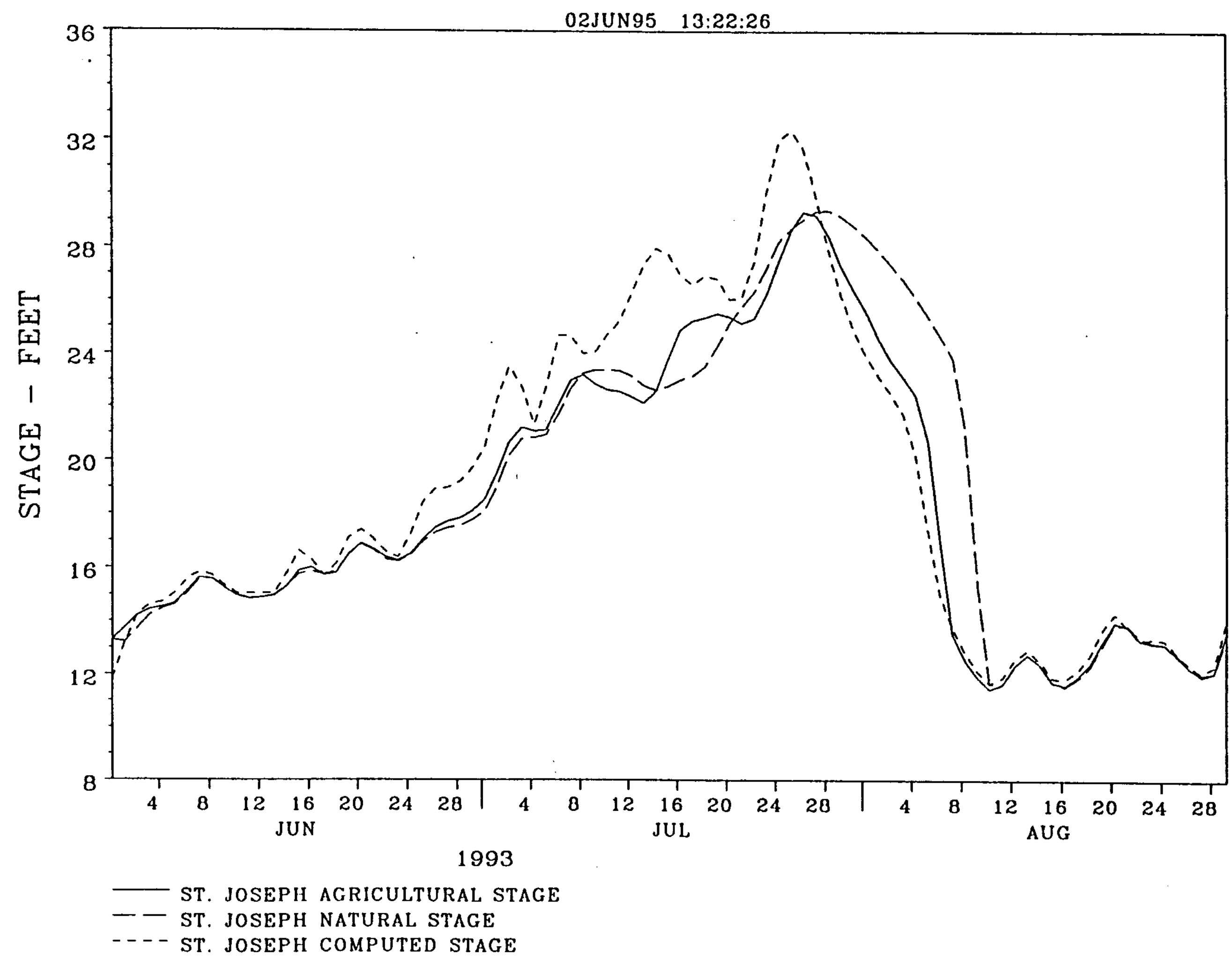
BOONVILLE GAUGE RATING CURVE


MISSOURI RIVER
ST JOSEPH - RM 448.2
COMPUTED VS OBSERVED STAGES — 1993 FLOOD
02JUN95 13:22:18
STAGE – FEET
8
12
16
20
24
28
32
36
4 8 12 16 20 24 28
JUN
4 8 12 16 20 24 28
JUL
4 8 12 16 20 24 28
AUG
1993
ST. JOSEPH COMPUTED STAGE
ST. JOSEPH OBSERVED STAGE

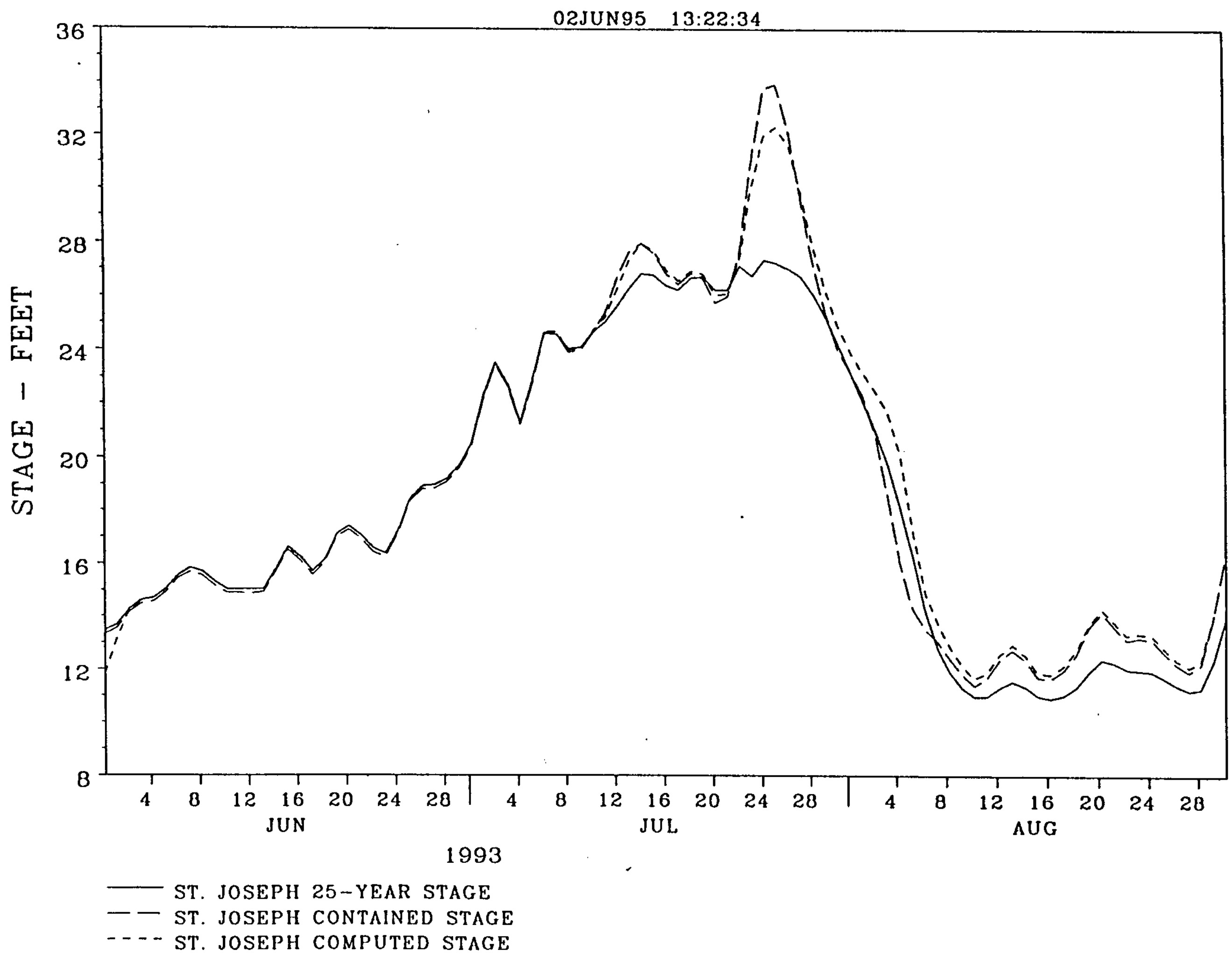
MISSOURI RIVER

ST JOSEPH - RM 448.2

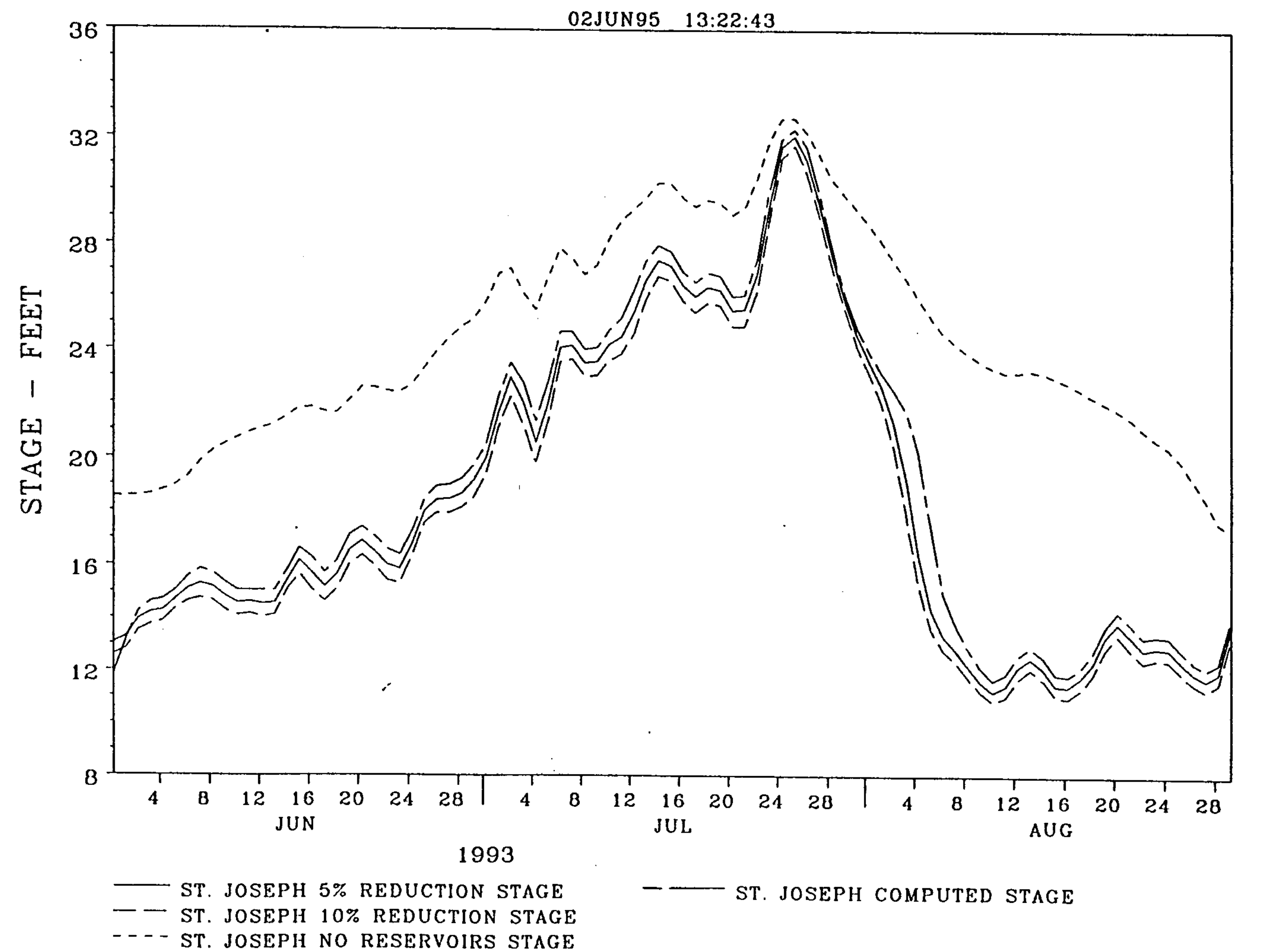
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS



MISSOURI RIVER
ST JOSEPH - RM 448.2
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



MISSOURI RIVER
ST JOSEPH - RM 448.2
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS



MISSOURI RIVER
ST JOSEPH - RM 448.2
LEVEE SETBACKS

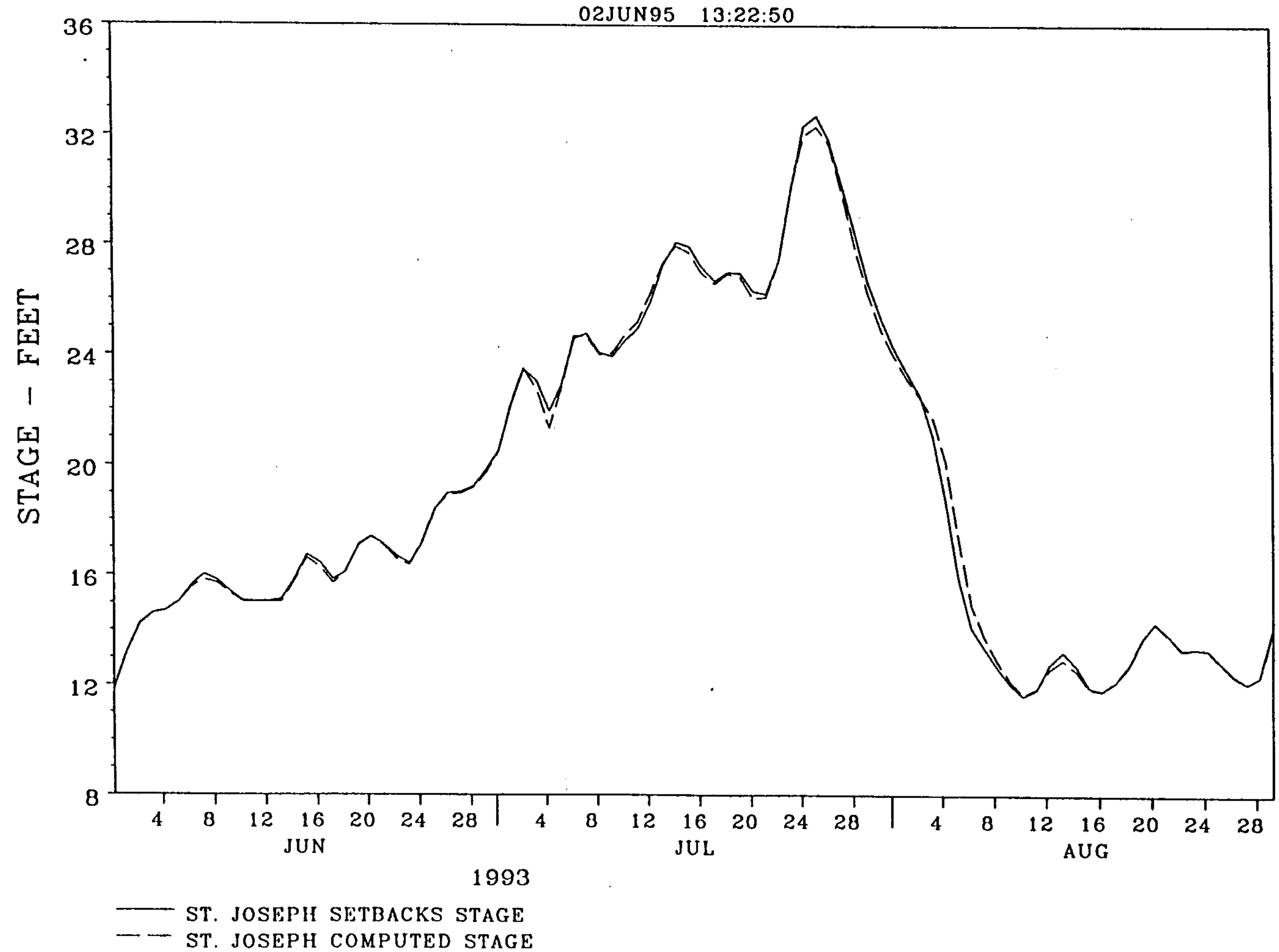


PLATE KC-3-E

MISSOURI RIVER
KANSAS CITY - RM 366.1
COMPUTED VS OBSERVED STAGES — 1993 FLOOD

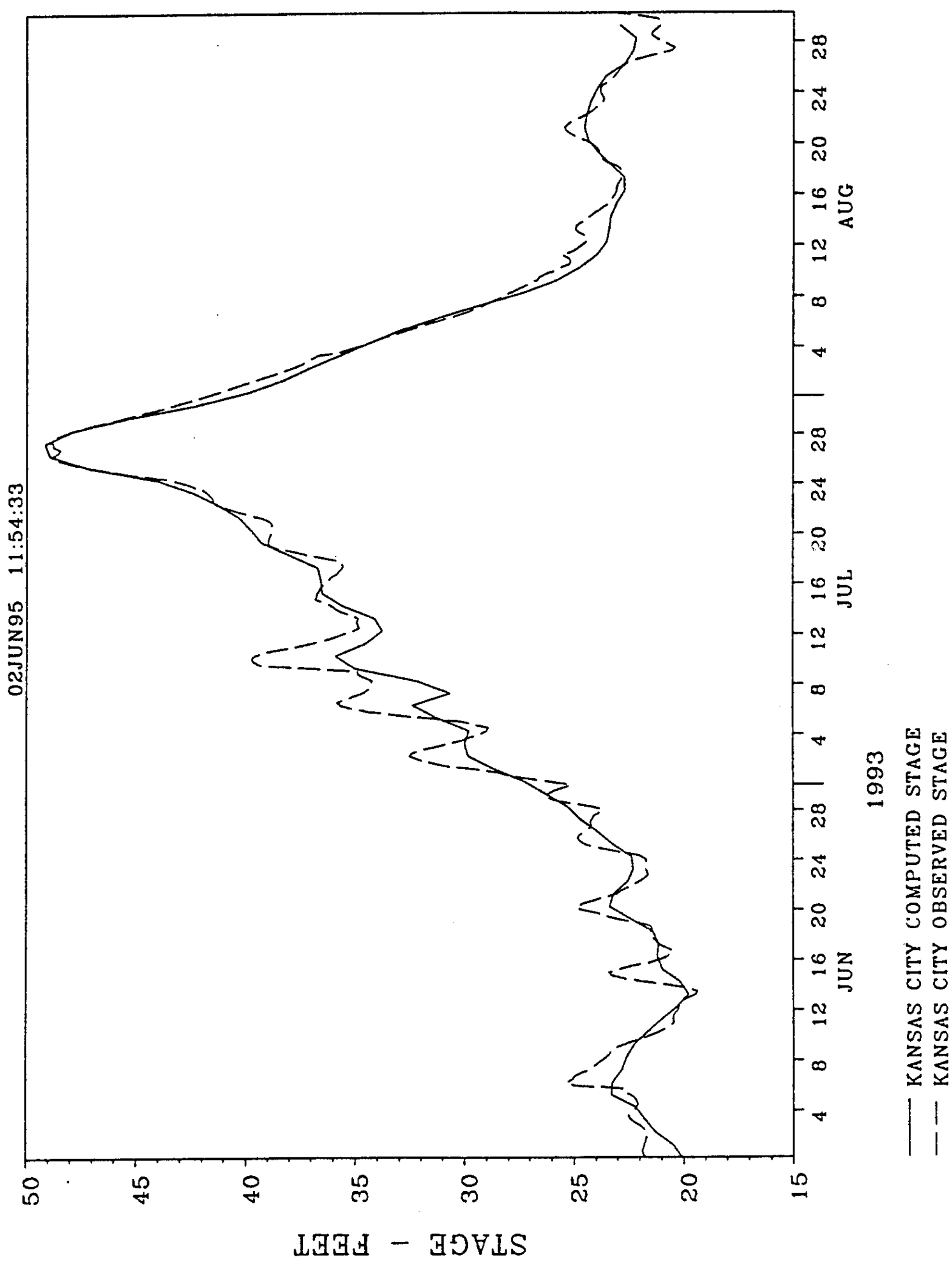


PLATE KC-4-A

MISSOURI RIVER

KANSAS CITY - RM 366.1

LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

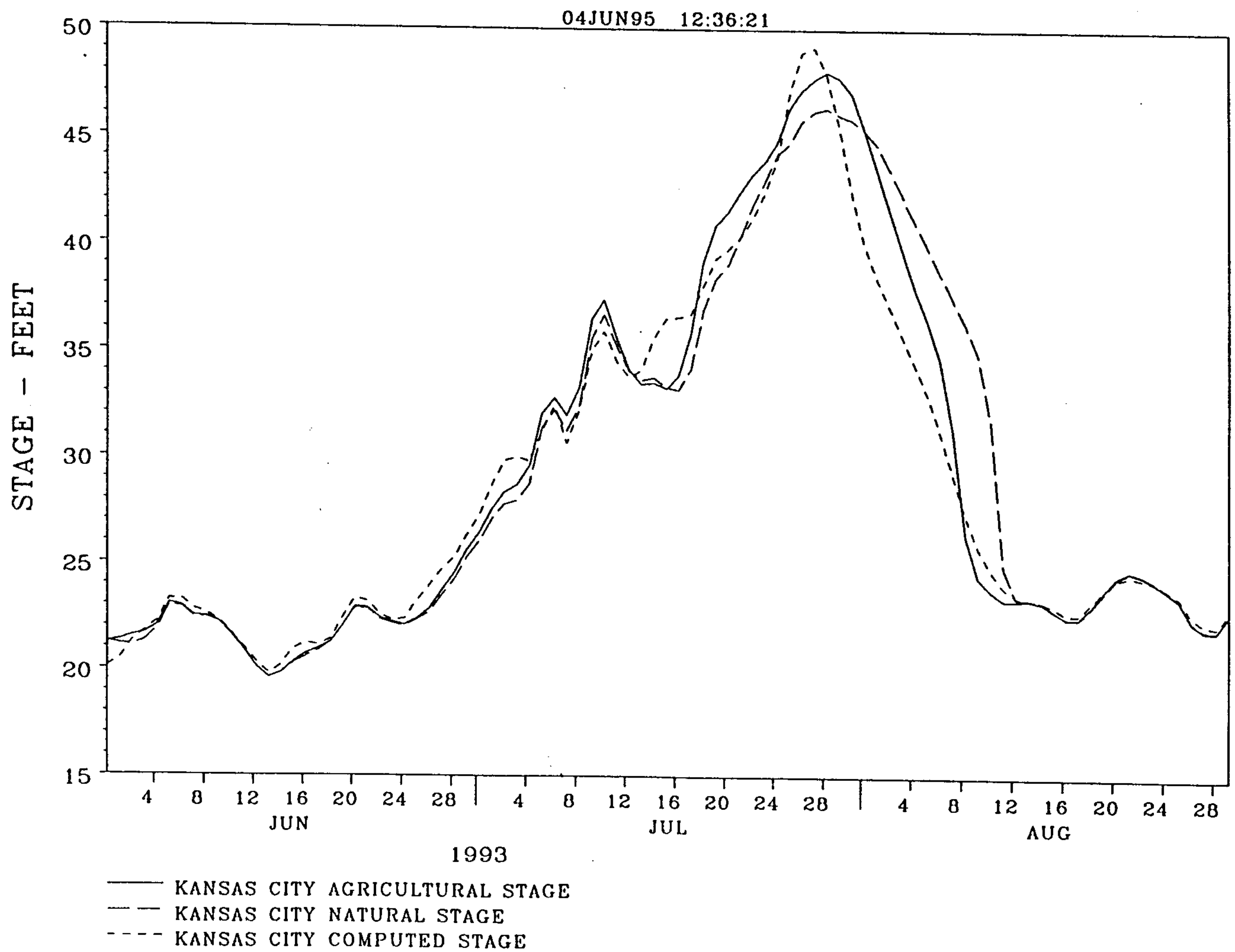
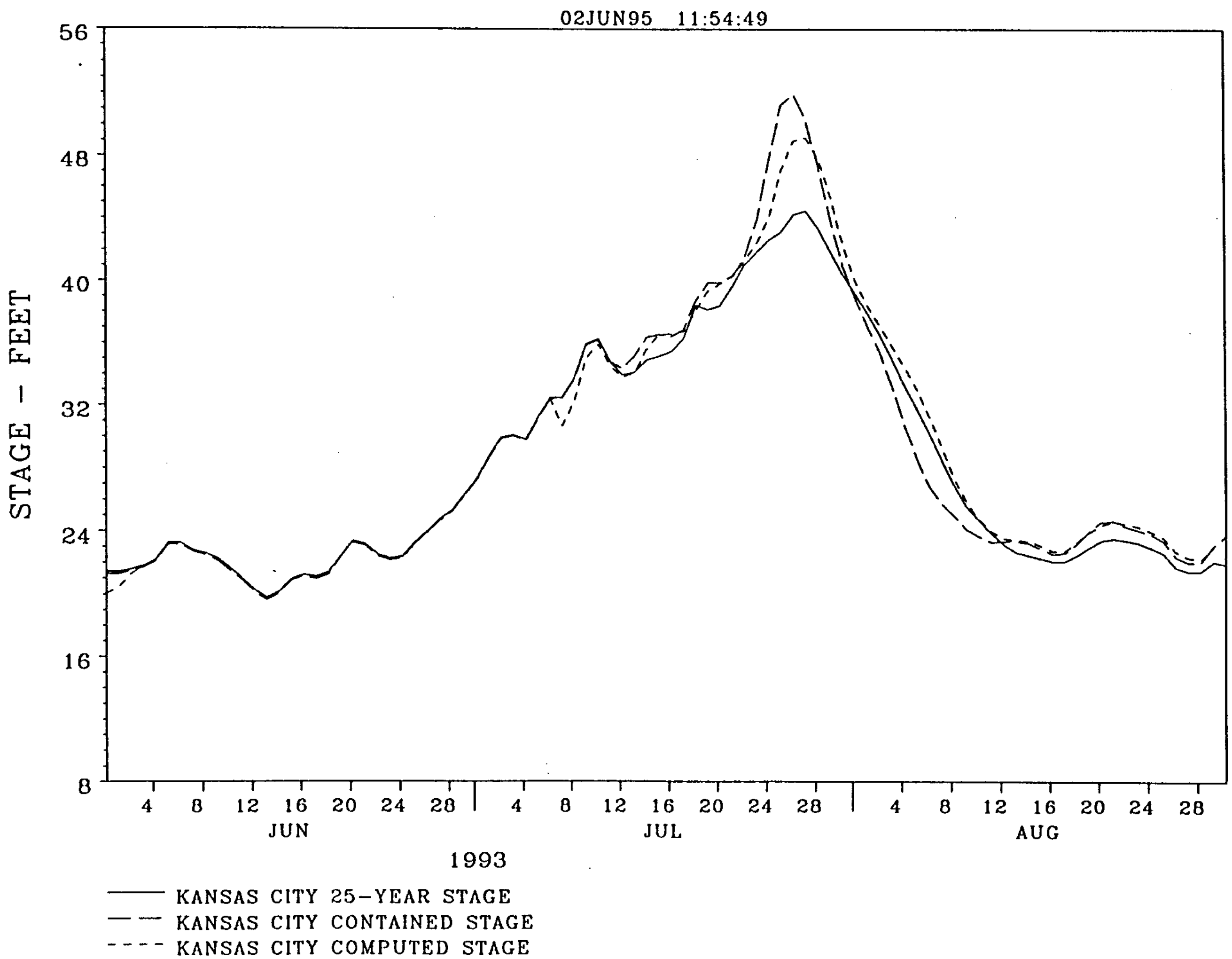
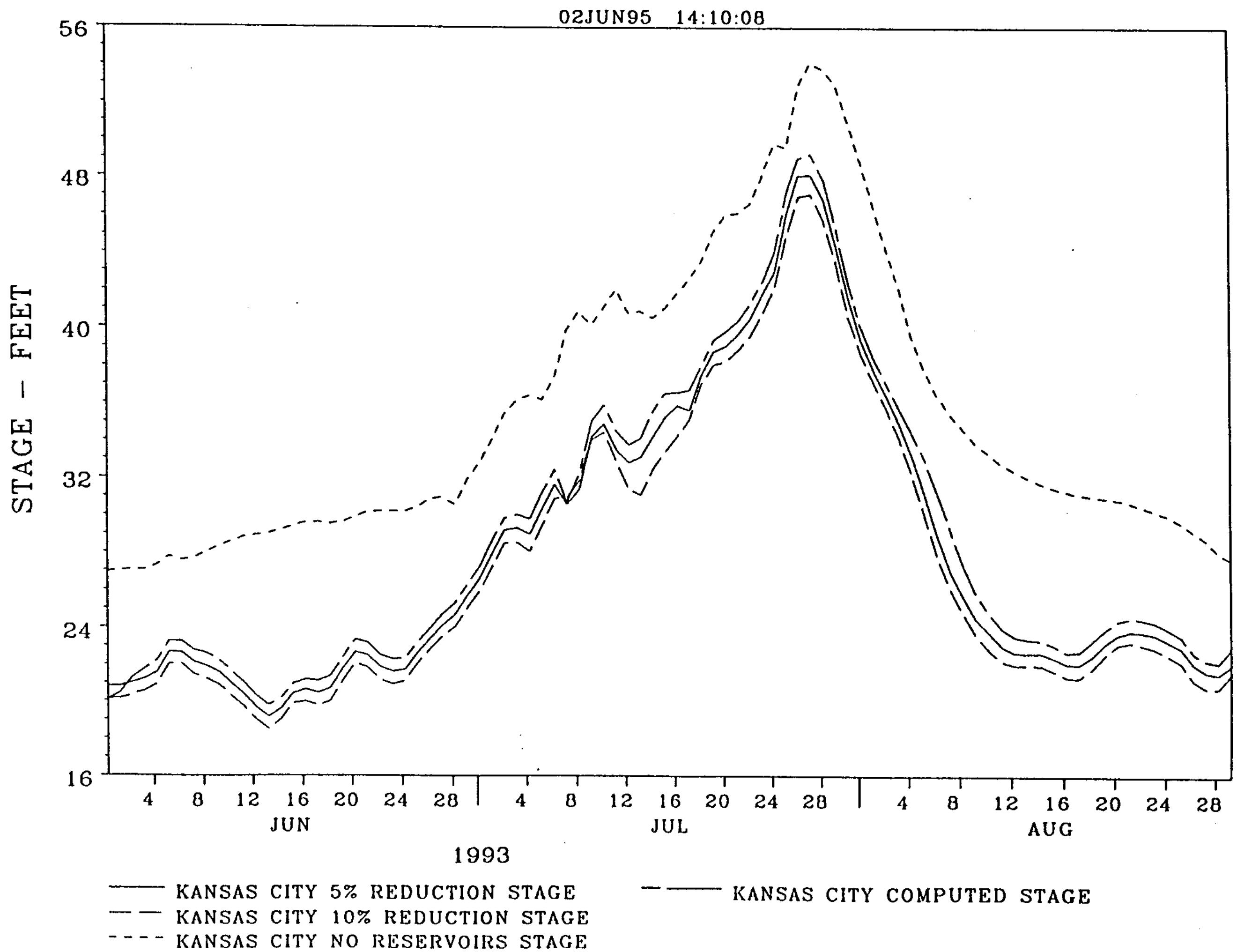


PLATE KC-4-B

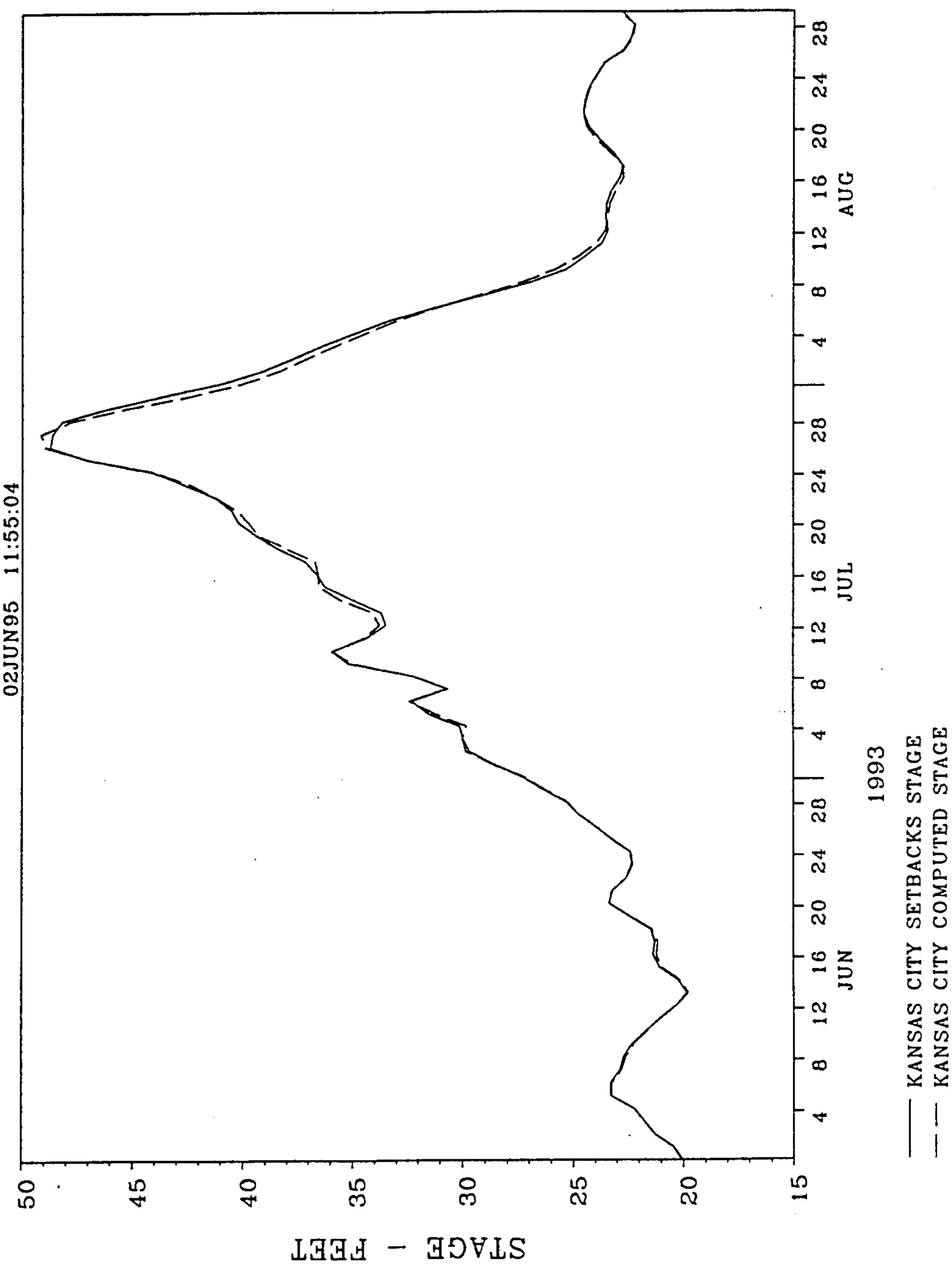
MISSOURI RIVER
KANSAS CITY - RM 366.1
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



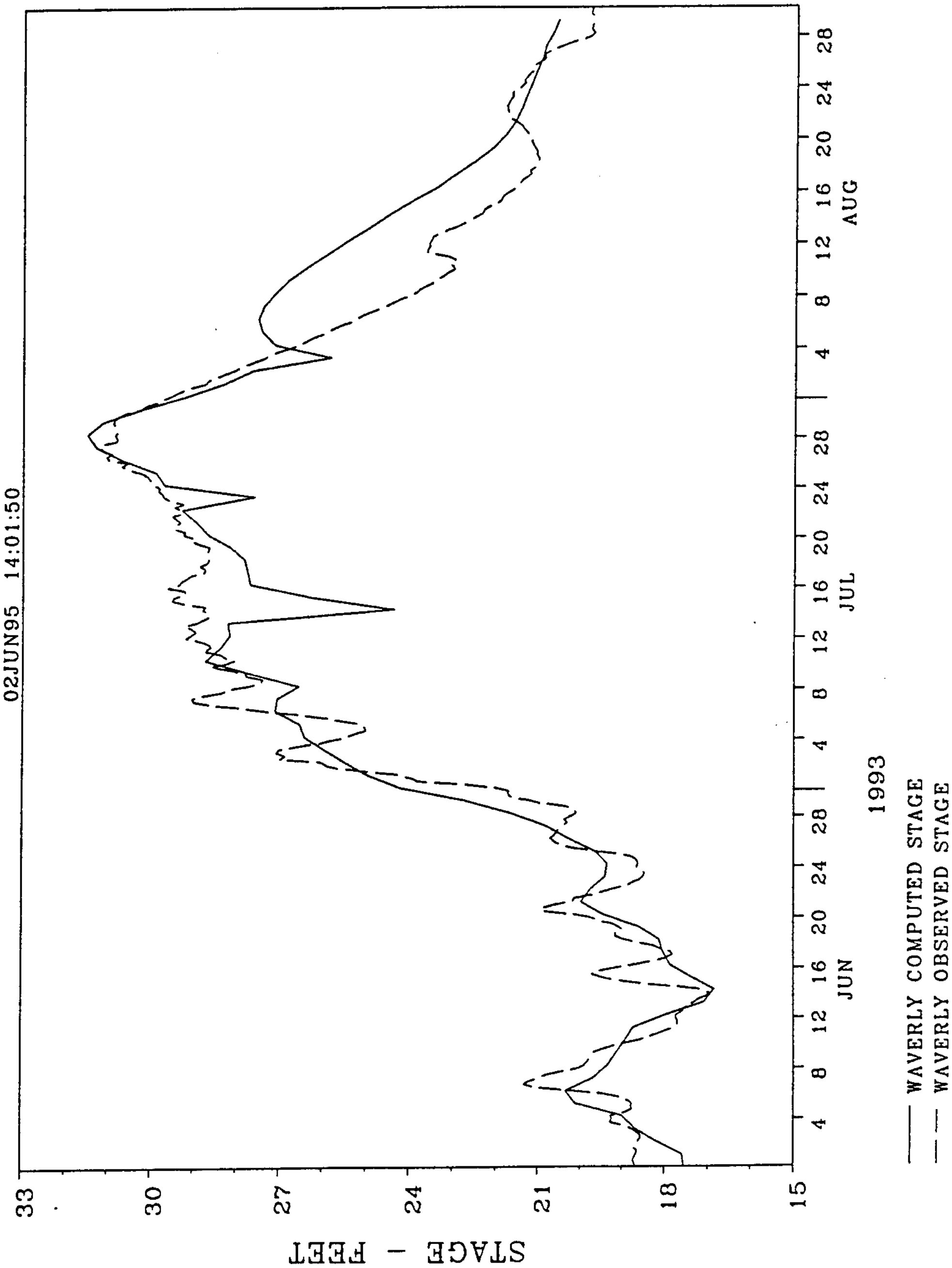
MISSOURI RIVER
KANSAS CITY - RM 366.1
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS



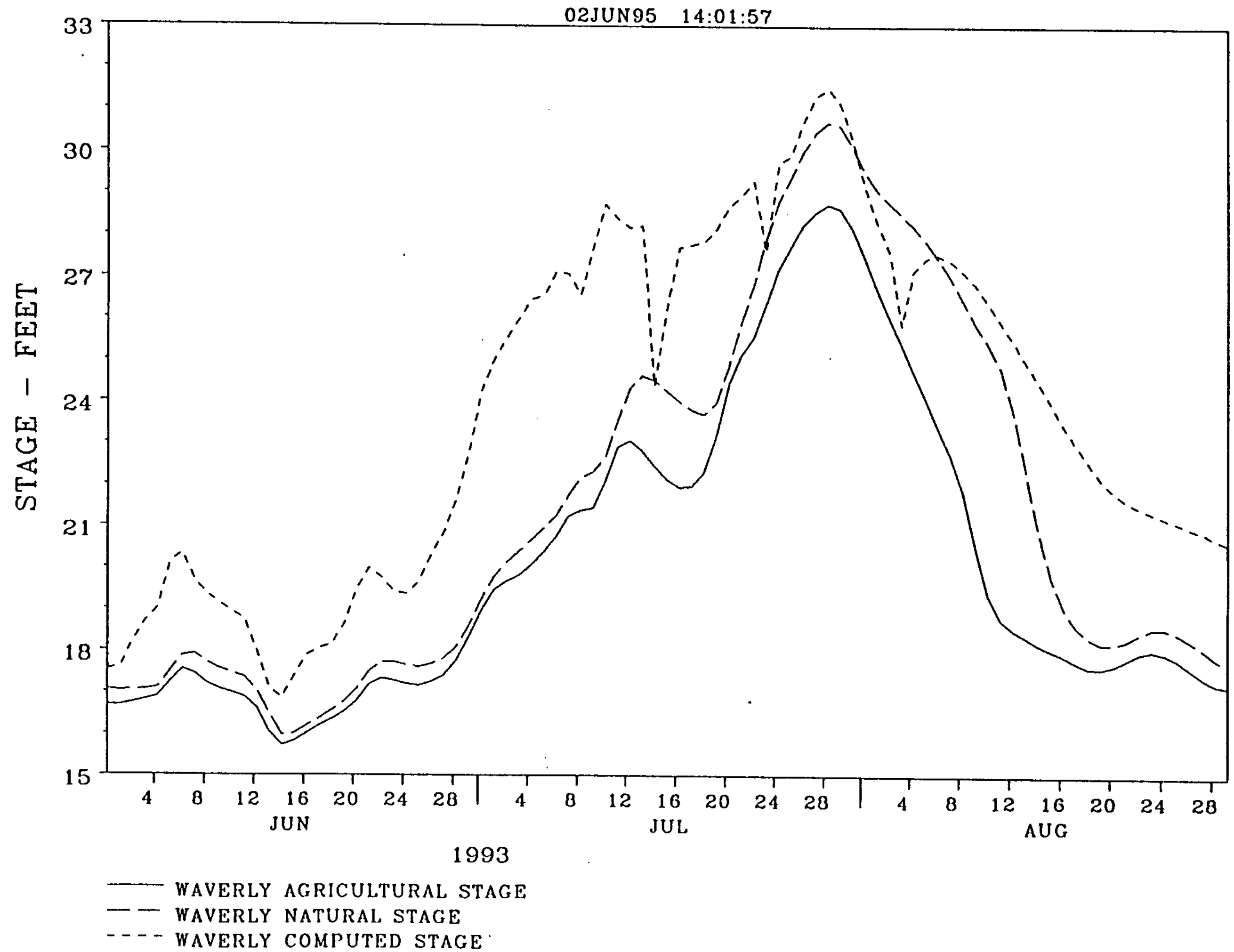
MISSOURI RIVER
KANSAS CITY - RM 366.1
LEVEE SETBACKS



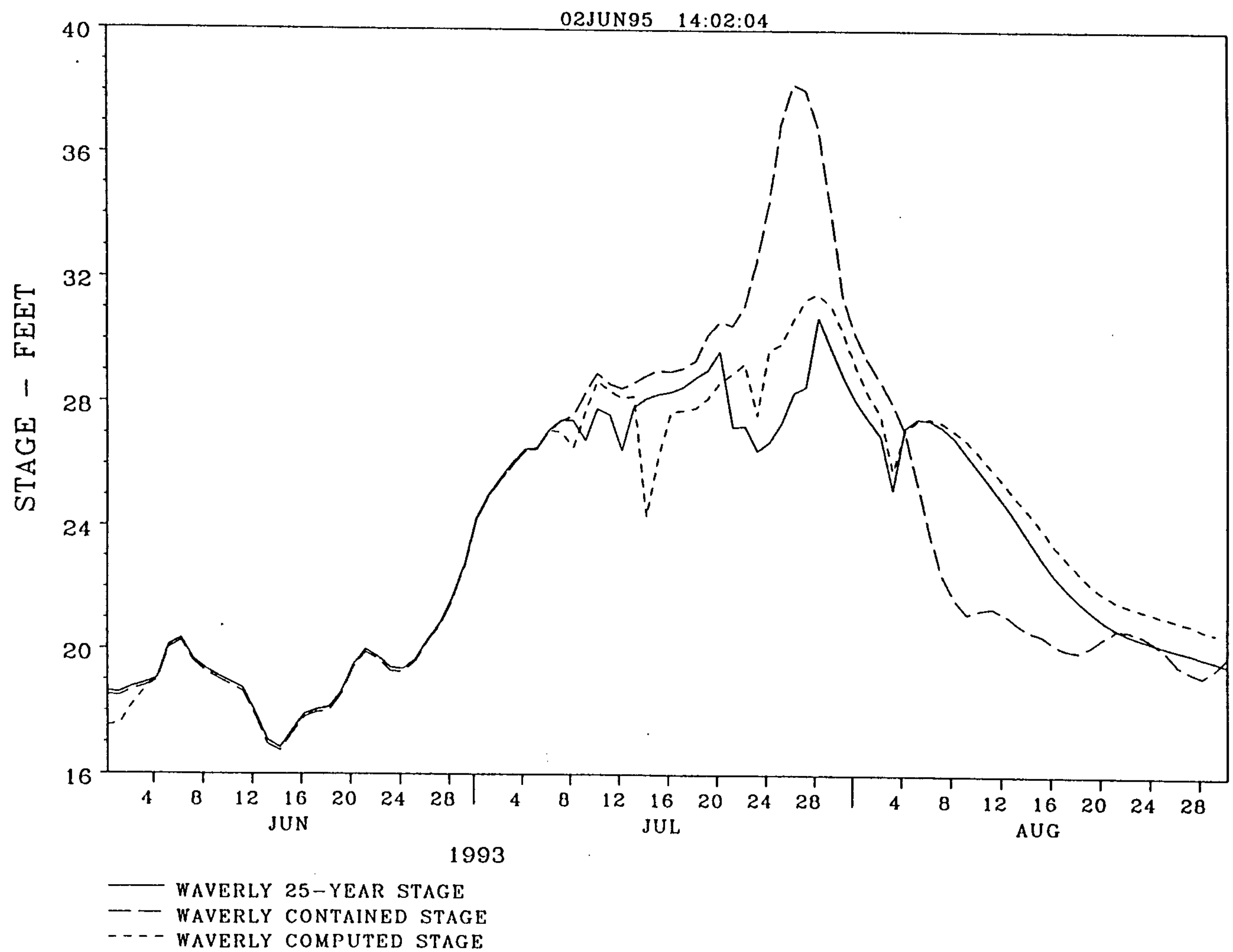
MISSOURI RIVER
WAVERLY - RM 293.4
COMPUTED VS OBSERVED STAGES — 1993 FLOOD



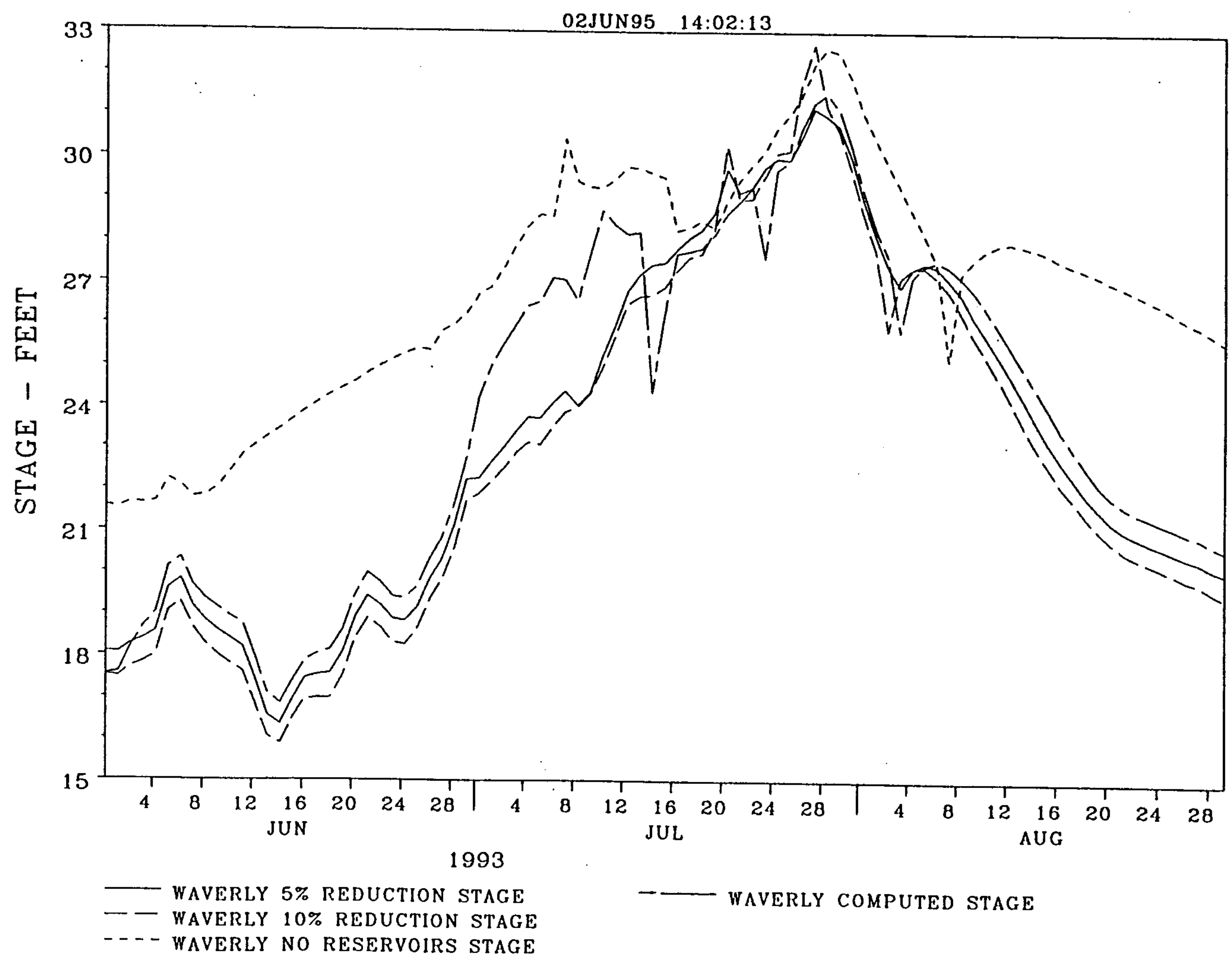
MISSOURI RIVER
WAVERLY - RM 293.4
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS



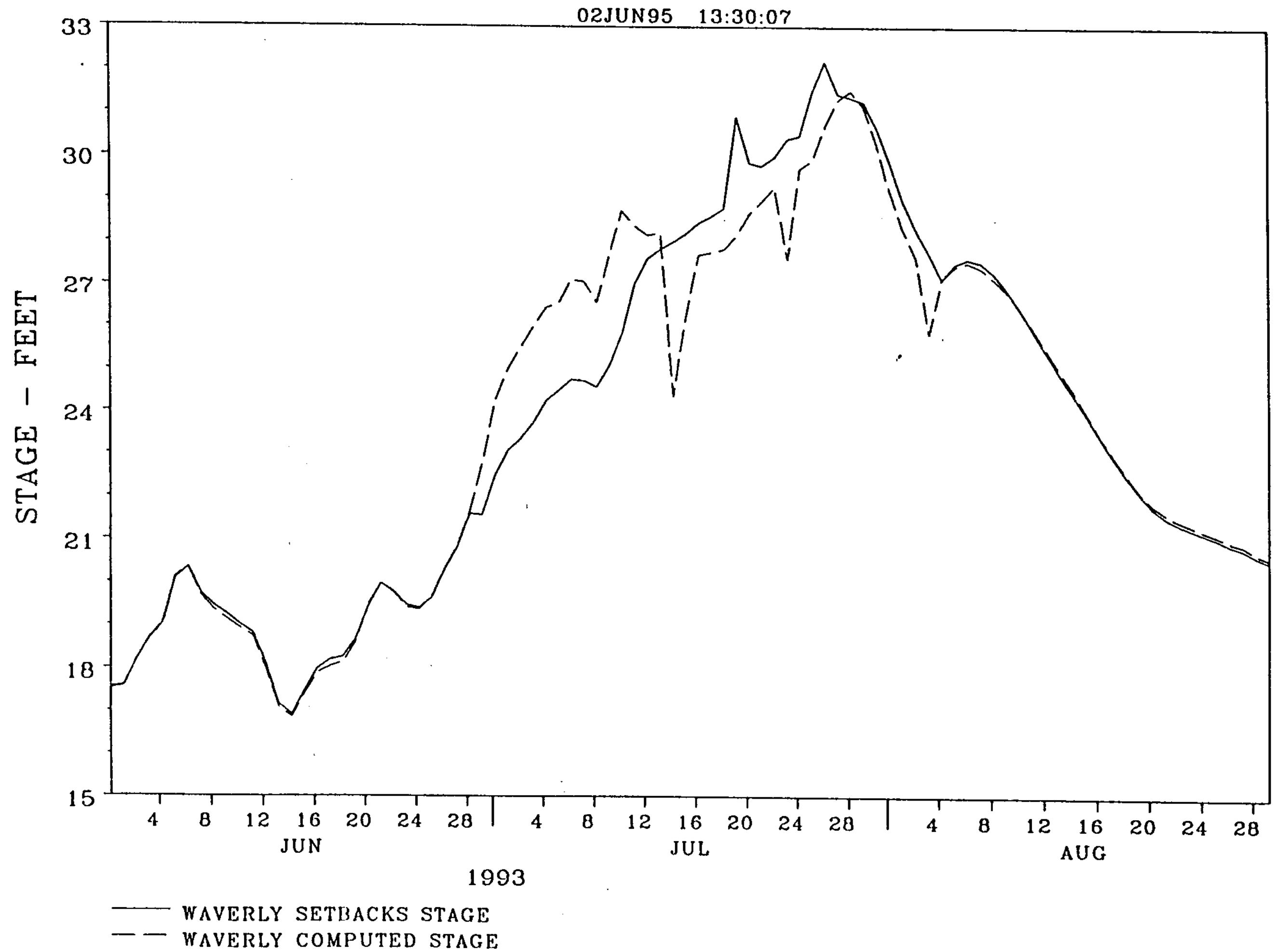
MISSOURI RIVER
WAVERLY - RM 293.4
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES

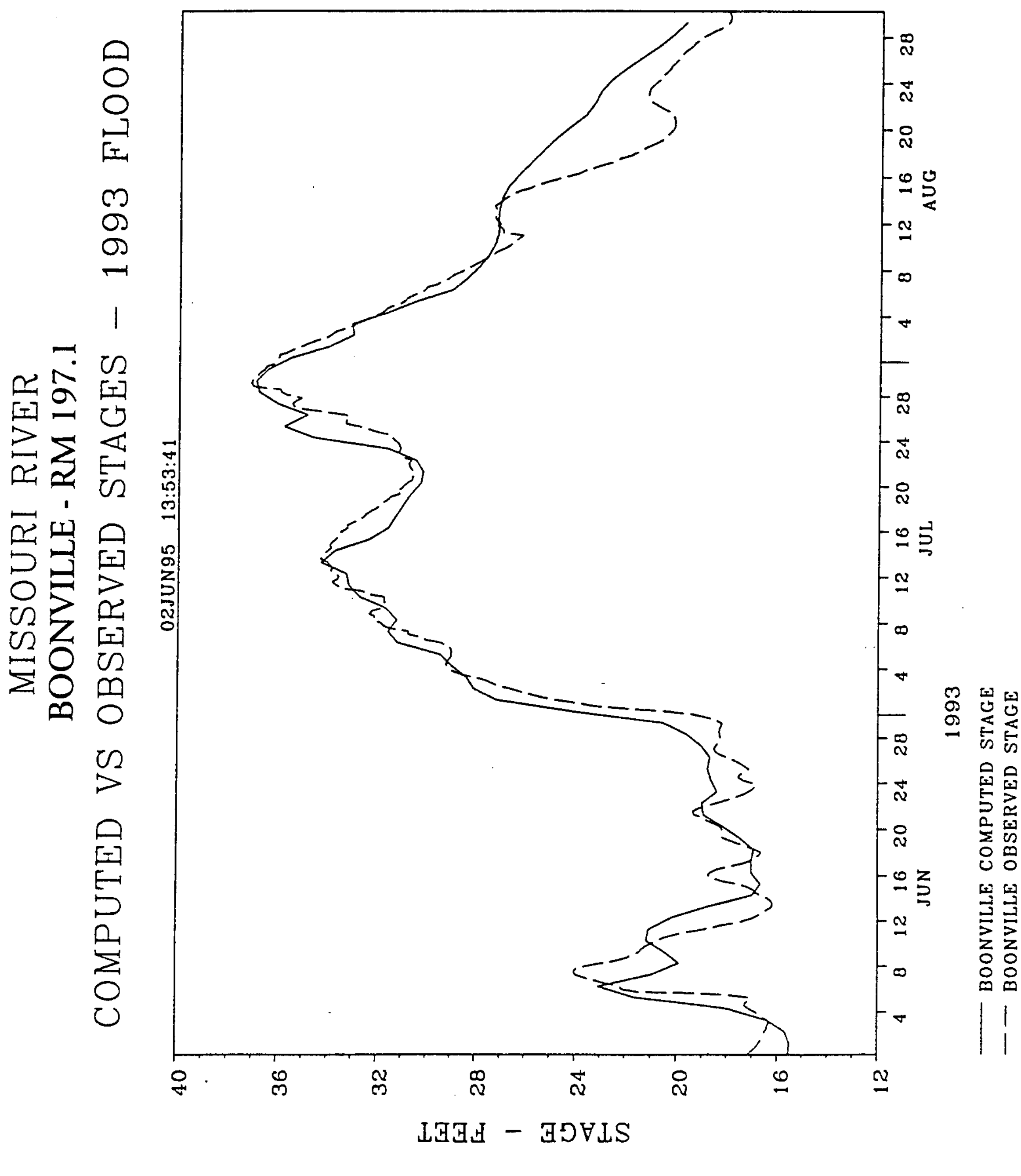


MISSOURI RIVER
WAVERLY - RM 293.4
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS



MISSOURI RIVER
WAVERLY - RM 293.4
LEVEE SETBACKS




MISSOURI RIVER
BOONVILLE - RM 197.1
COMPUTED VS OBSERVED STAGES — 1993 FLOOD
02JUN95 13:53:41
STAGE - FEET
40
36
32
28
24
20
16
12
4 8 12 16 20 24 28
JUN
4 8 12 16 20 24 28
JUL
4 8 12 16 20 24 28
AUG
1993
BOONVILLE COMPUTED STAGE
BOONVILLE OBSERVED STAGE

MISSOURI RIVER
BOONVILLE - RM 197.1
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

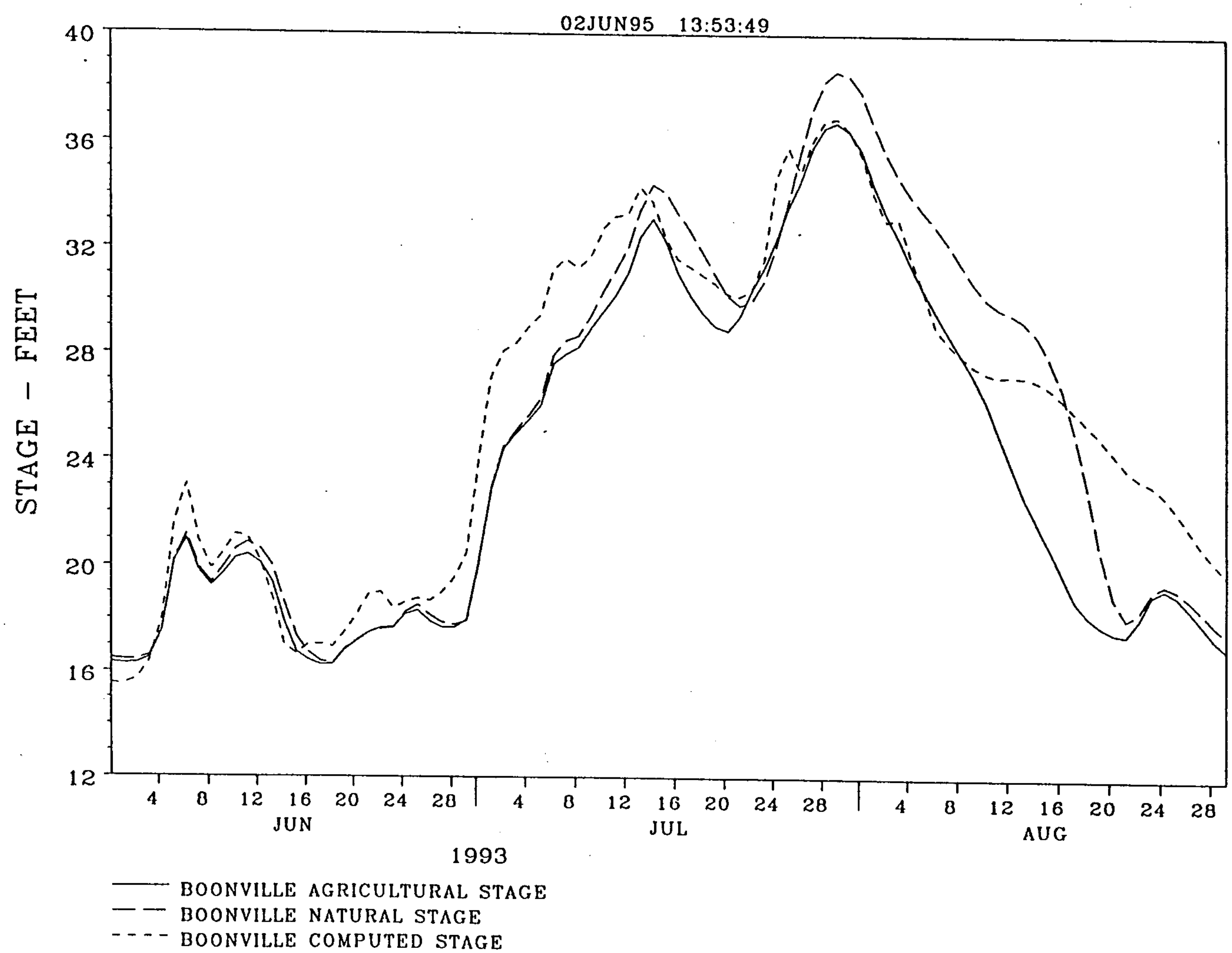
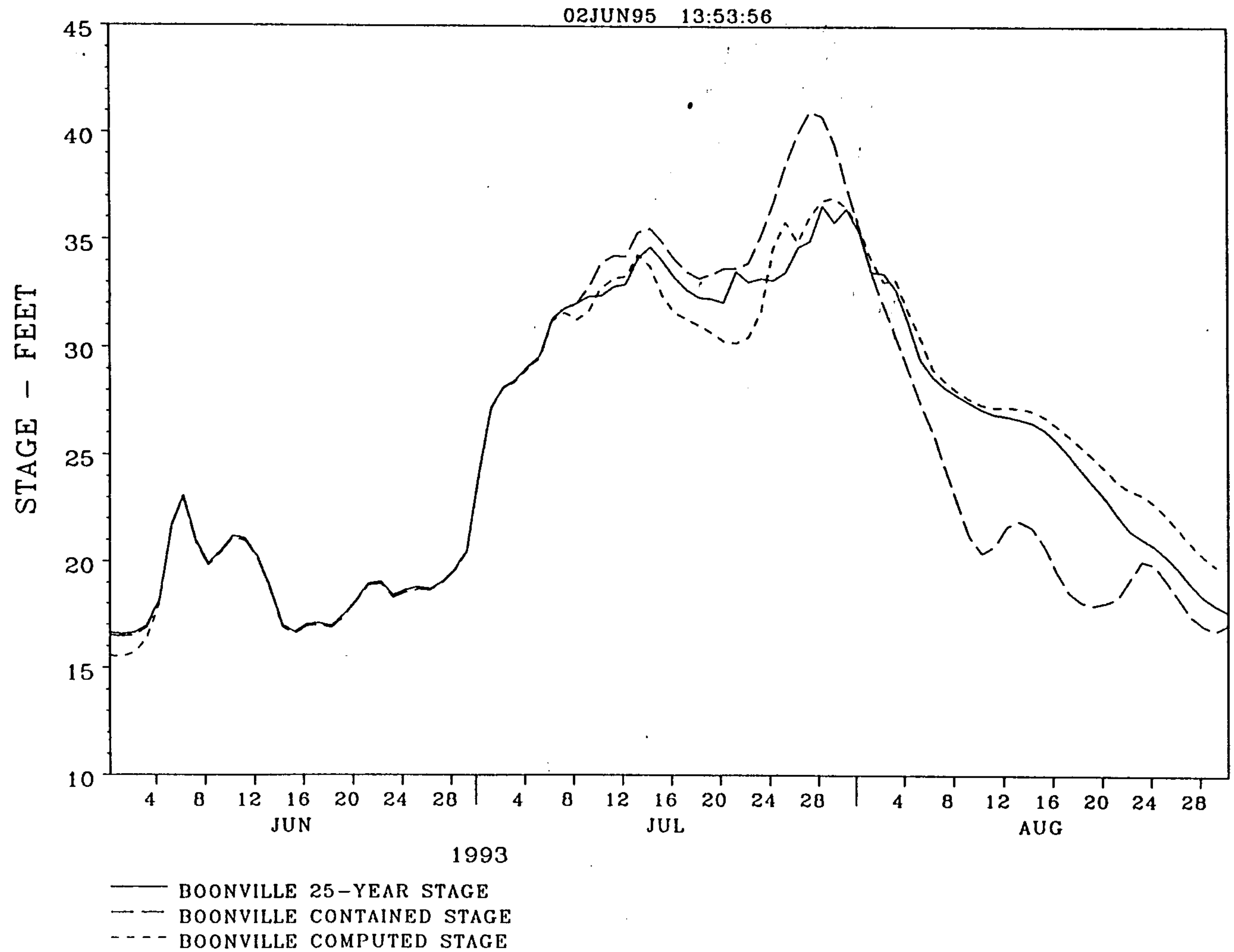
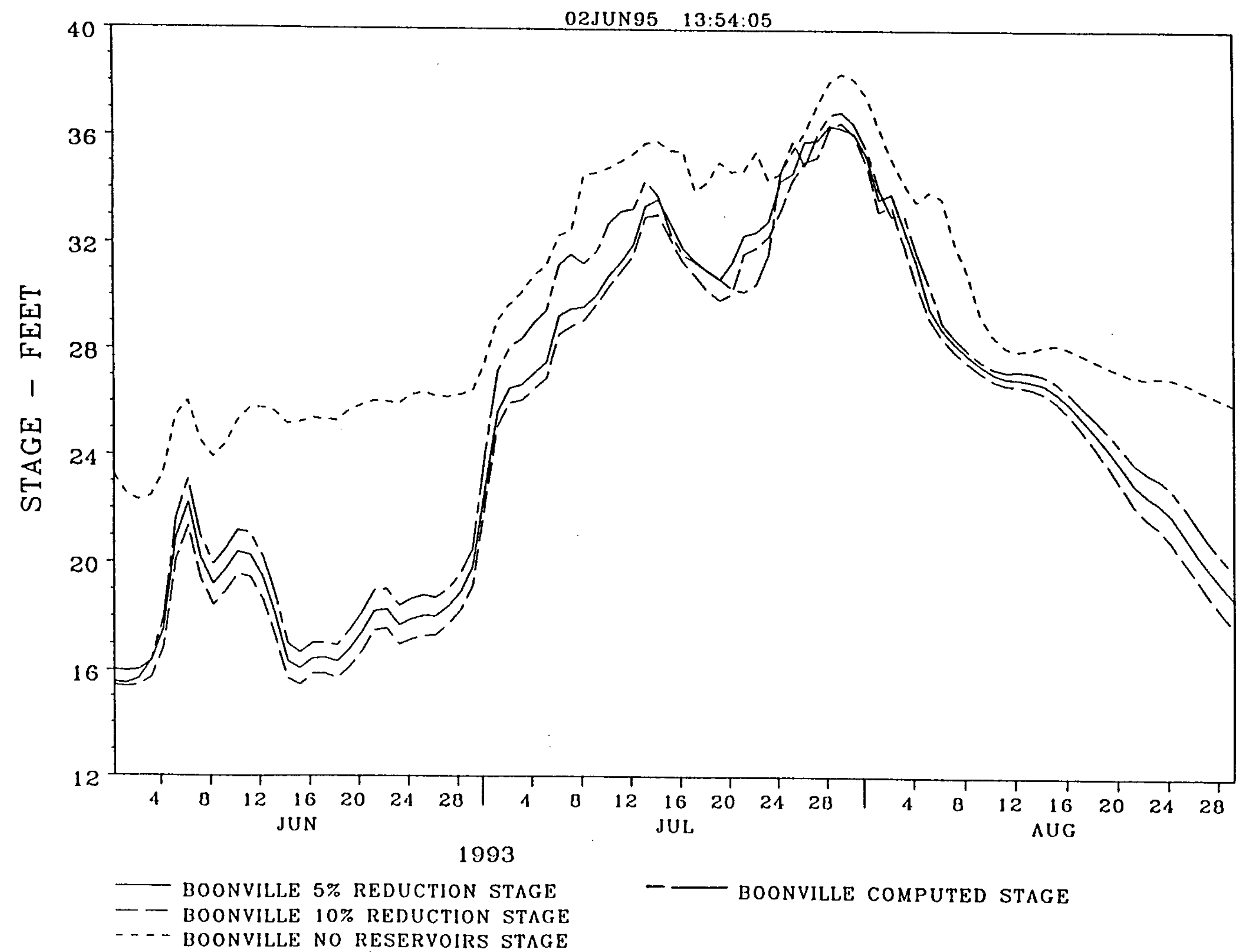


PLATE KC-6-B

MISSOURI RIVER
BOONVILLE - RM 197.1
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



MISSOURI RIVER
BOONVILLE - RM 197.1
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS



MISSOURI RIVER
BOONVILLE - RM 197.1
LEVEE SETBACKS

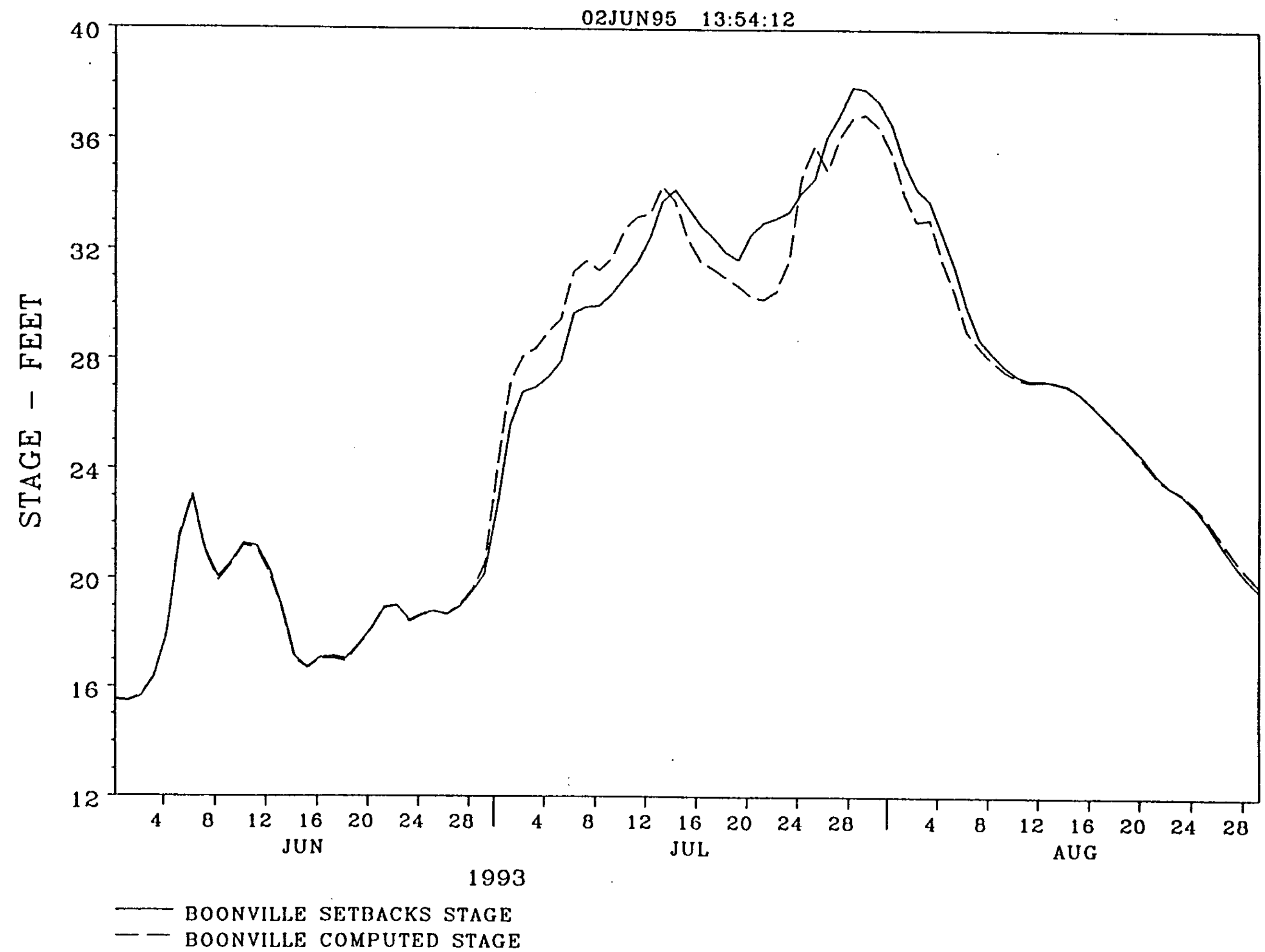
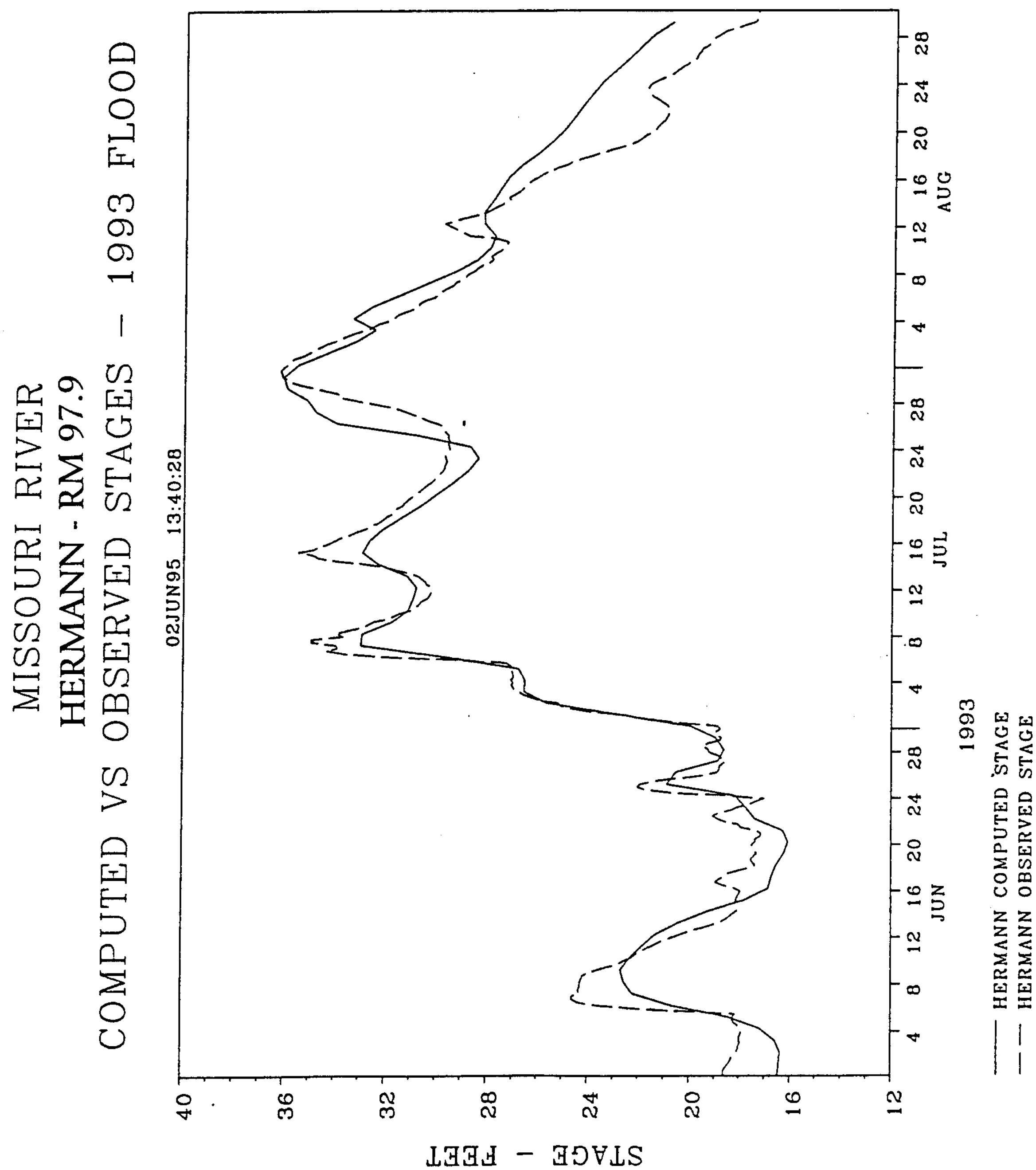


PLATE KC-6-E


MISSOURI RIVER
HERMANN - RM 97.9
COMPUTED VS OBSERVED STAGES – 1993 FLOOD
02JUN95 13:40:28
STAGE – FEET
40
36
32
28
24
20
16
12
4 8 12 16 20 24 28 4 8 12 16 20 24 28 4 8 12 16 20 24 28
JUN
JUL
AUG
1993
HERMANN COMPUTED STAGE
HERMANN OBSERVED STAGE

MISSOURI RIVER

HERMANN - RM 97.9

LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

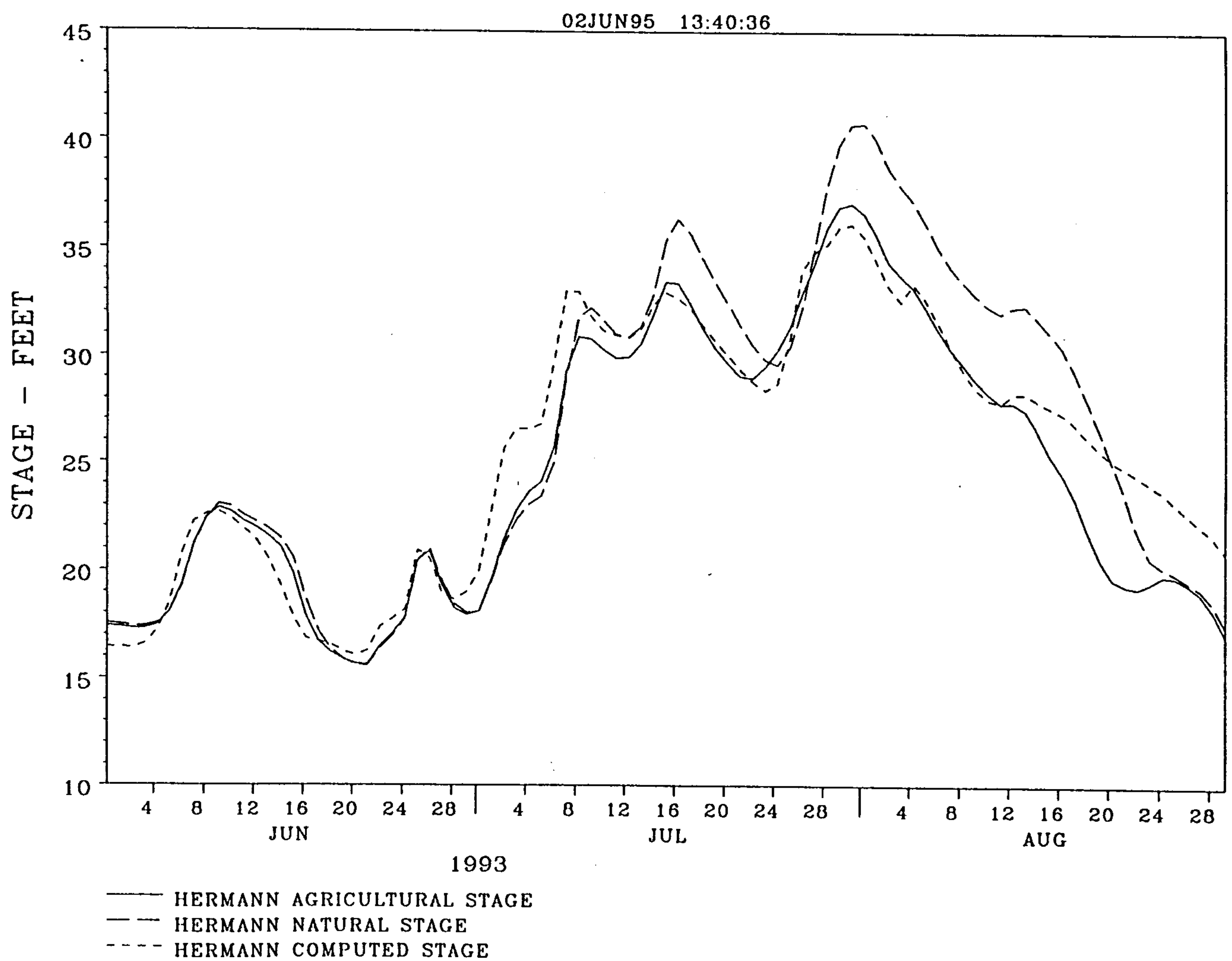


PLATE KC-7-B

MISSOURI RIVER
HERMANN - RM 97.9
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES

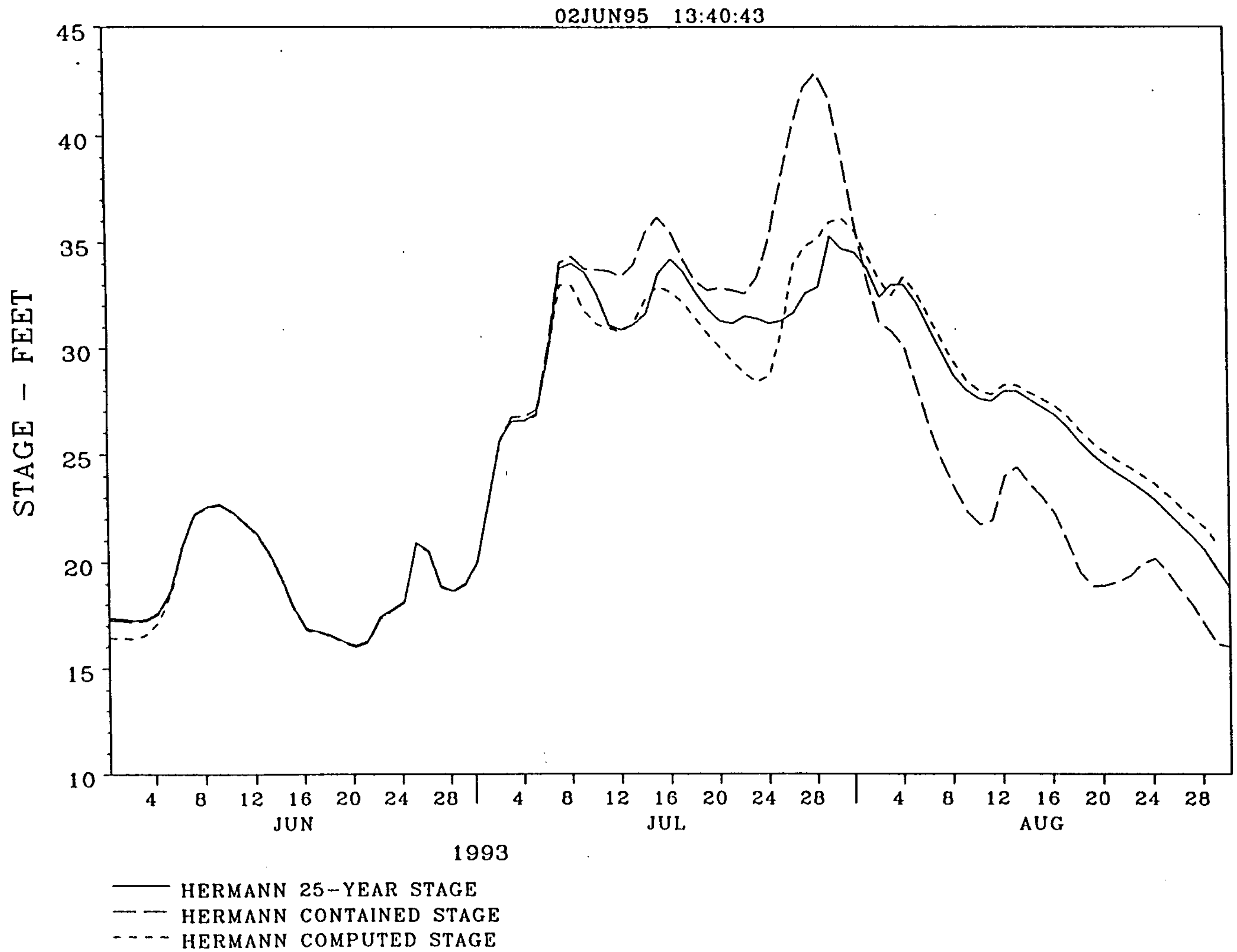


PLATE KC-7-C

MISSOURI RIVER
HERMANN - RM 97.9
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS

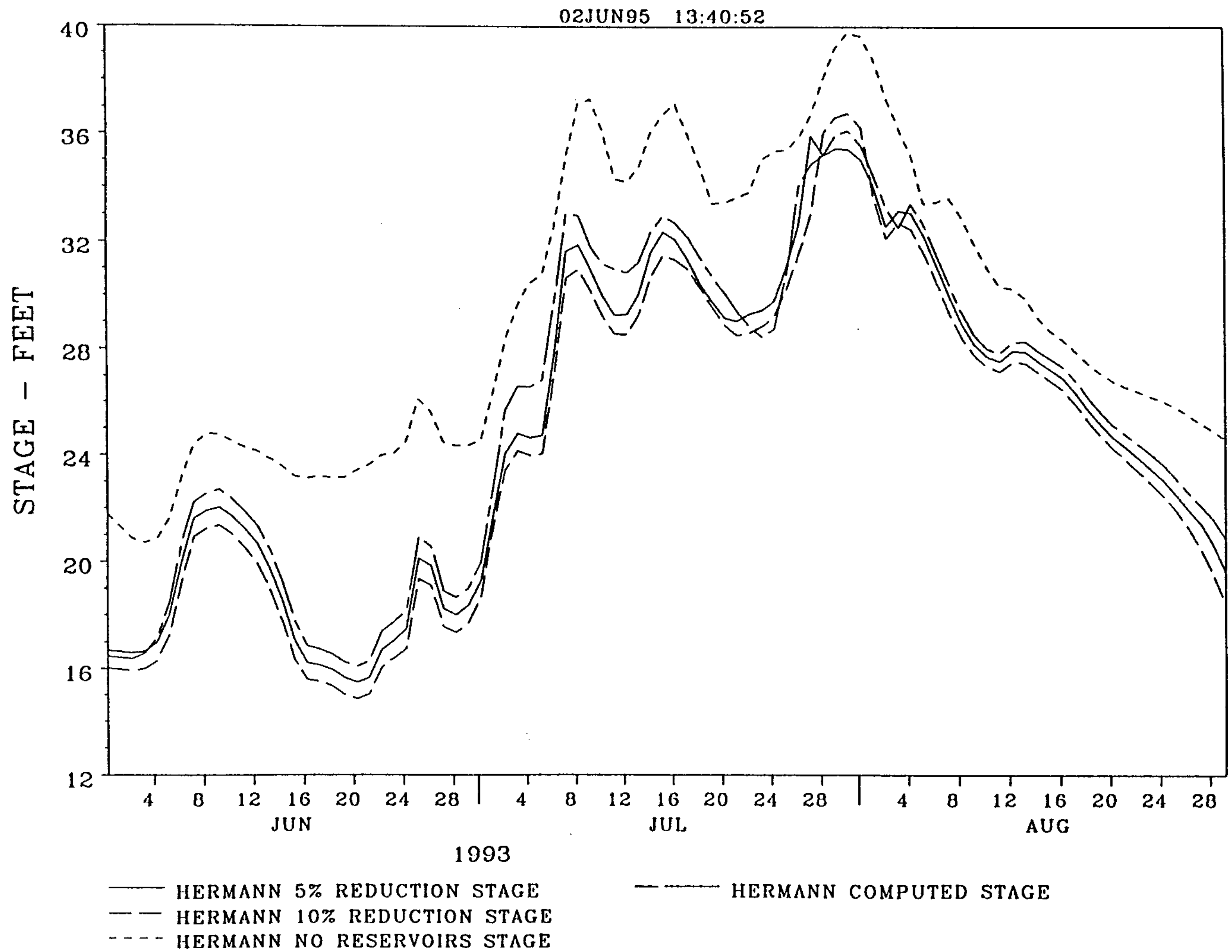
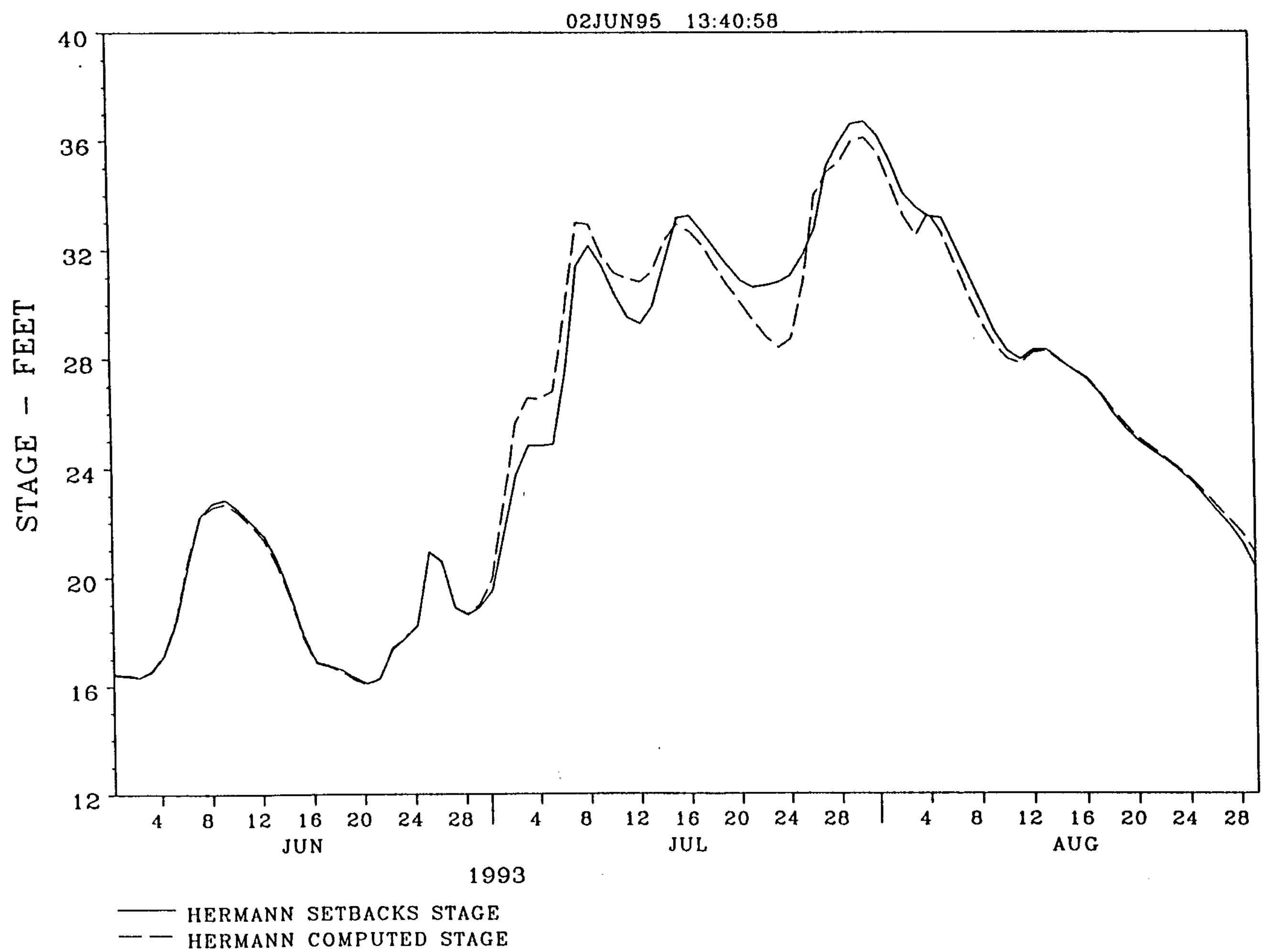


PLATE KC-7-D

MISSOURI RIVER
HERMANN - RM 97.9
LEVEE SETBACKS



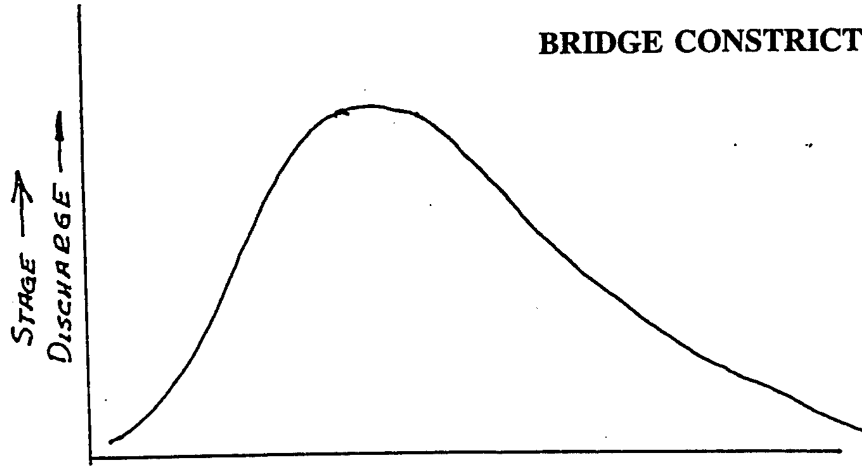


FIG A

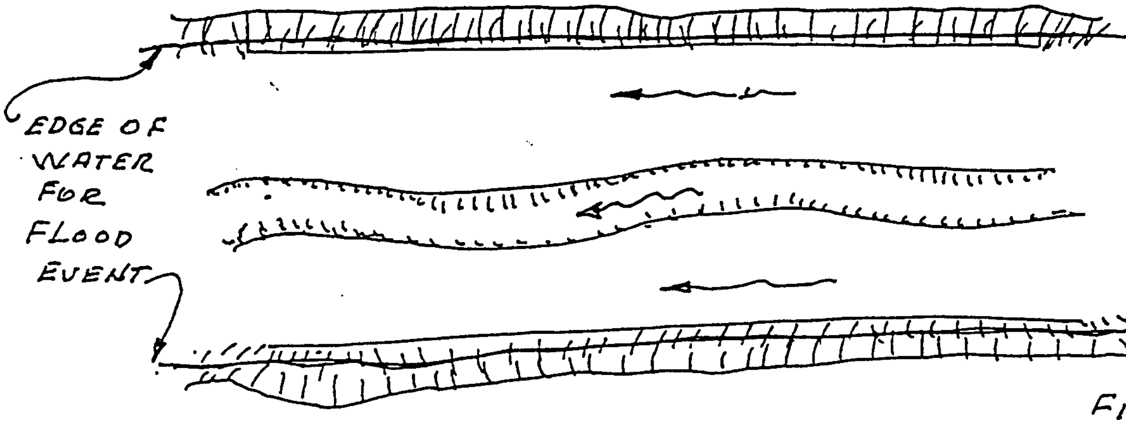


FIG B

PLAN VIEW

NOTE:
MAXIMUM ENERGY = depth + $\bar{V}^2/2g$ (No other energy available)

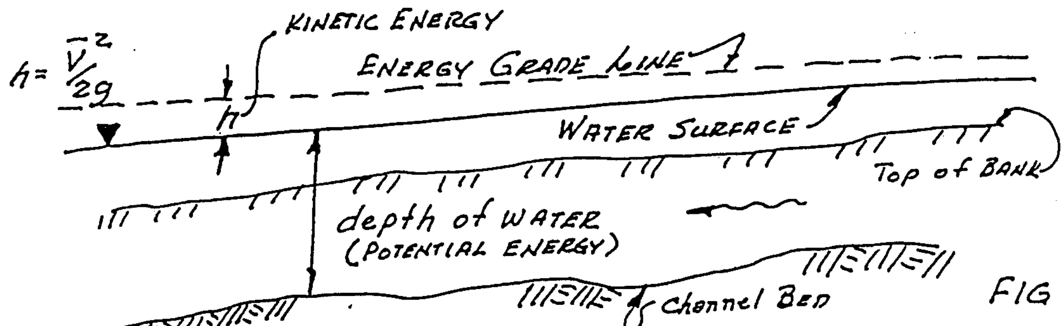


FIG C

PROFILE

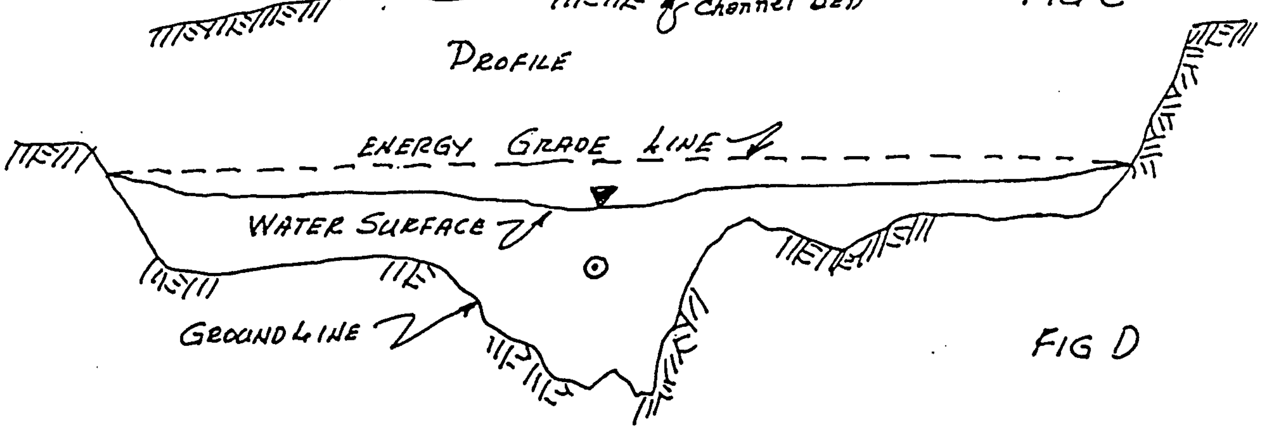
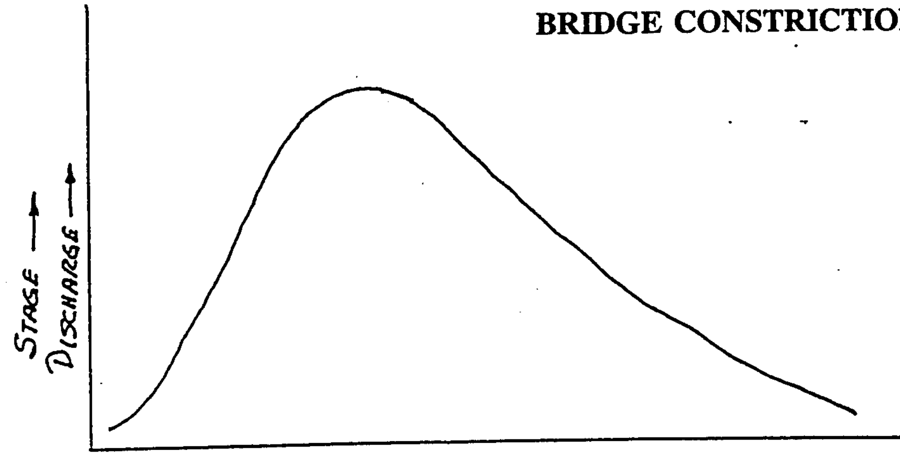


FIG D

CROSS SECTION



TIME →
FLOOD Hydrograph

FIG A
SAME AS FIG A
Sheet 1

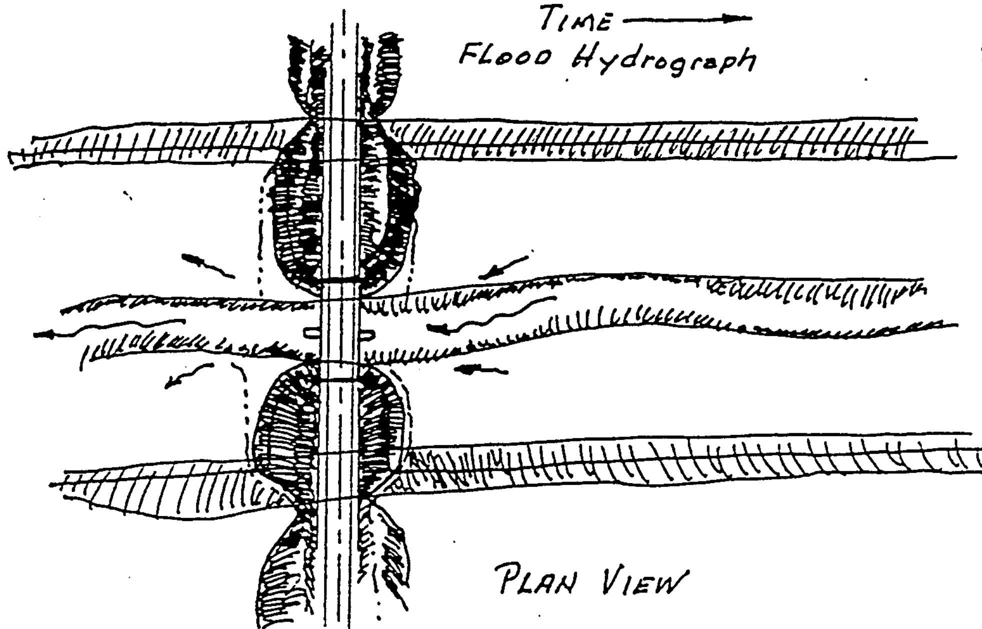


FIG B

PLAN VIEW

NOTE:
MAXIMUN ENERGY = depth + $\bar{V}^2/2g$ (No other energy available)

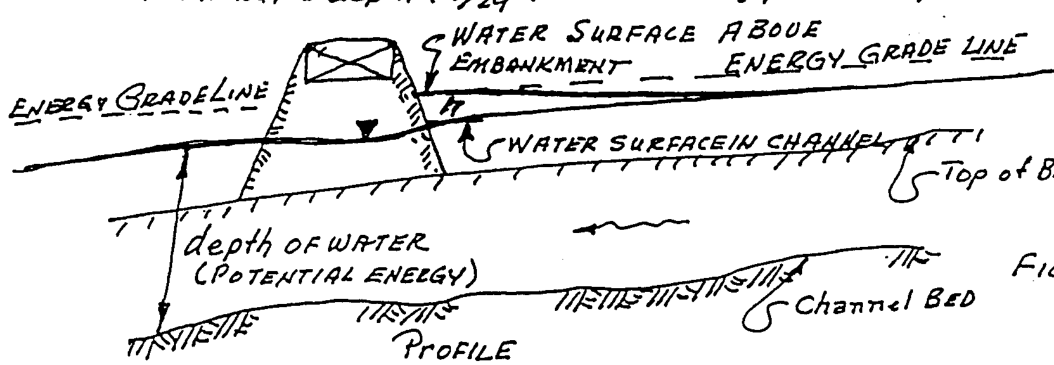


FIG C

PROFILE

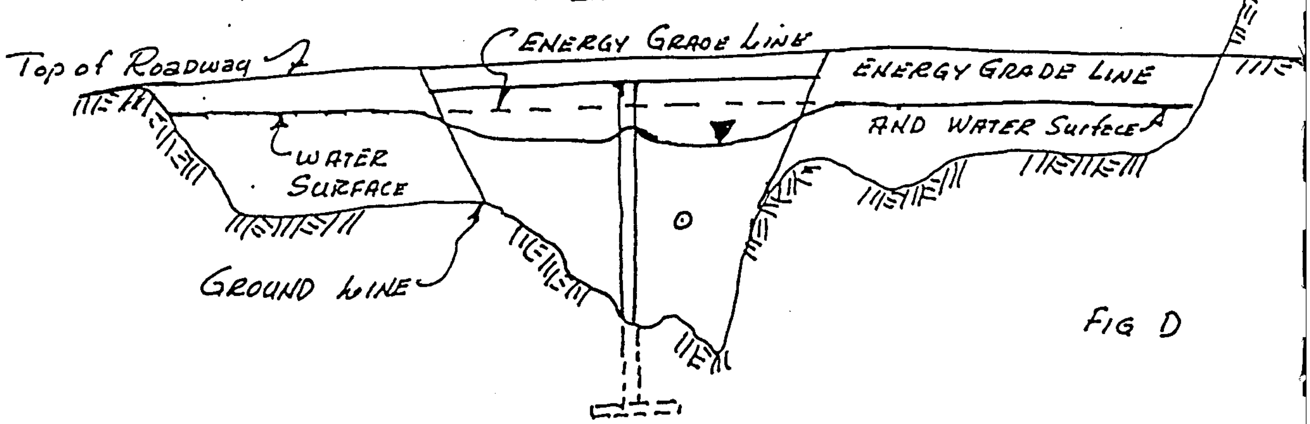
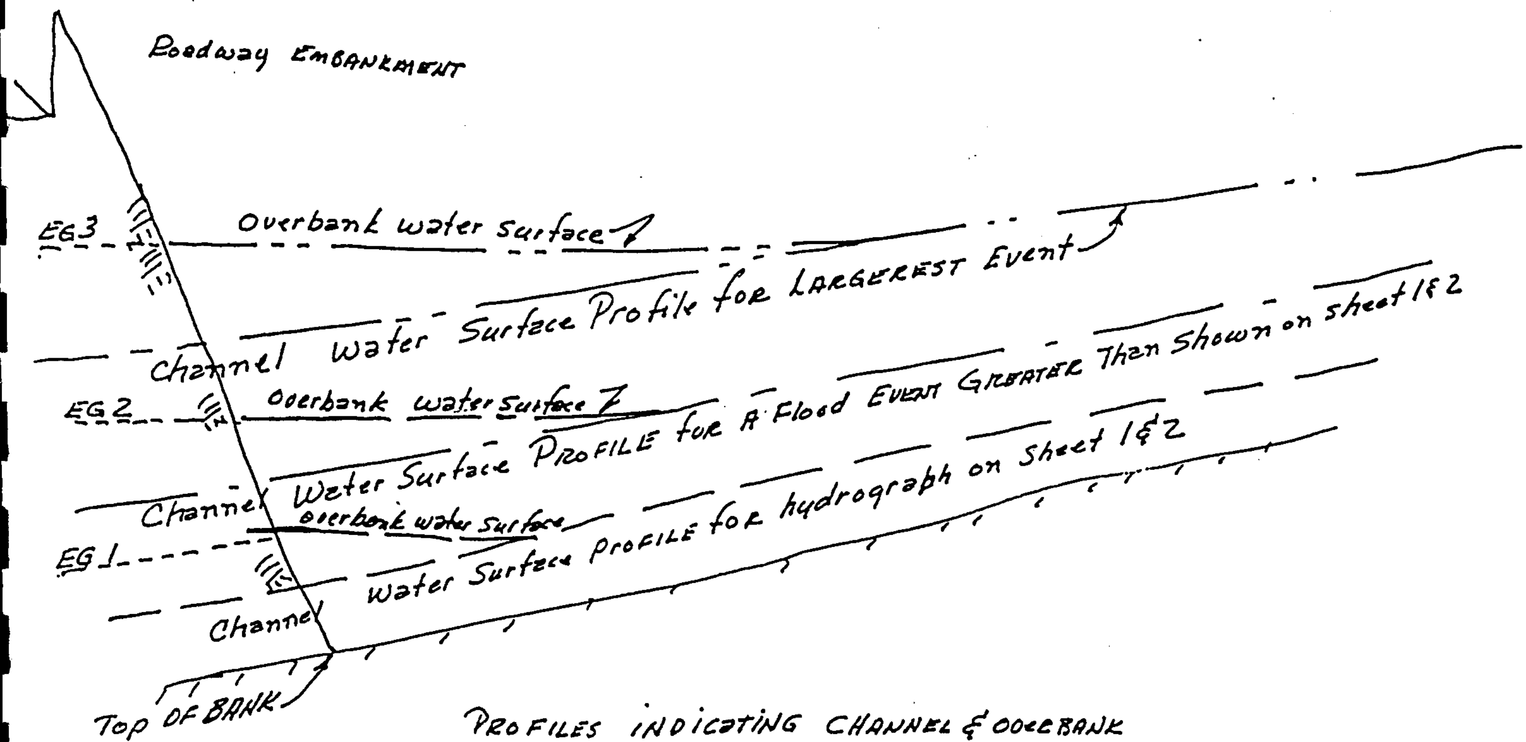


FIG D

CROSS SECTION ABOVE BRIDGE



PROFILES INDICATING CHANNEL & OVERBANK WATER SURFACE PROFILES FOR DIFFERENT FLOOD EVENTS

EG1 = ENERGY FOR SMALLEST FLOOD
EG2 = " " LARGER FLOOD
EG3 = " " LARGEST FLOOD

TABLE KC-1
MISSOURI RIVER
OBSERVED AND COMPUTED PEAK STAGES
1993 FLOOD EVENT

GAUGE LOCATION	RIVER MILE	GAUGE ZERO ELEVATION (FEET, N.G.V.D.)	FLOOD STAGE (FEET)	1993 OBSERVED PEAK STAGE (FEET)	1993 COMPUTED PEAK STAGE (FEET)
ST. JOSEPH, MO	448.2	788.2	17.0	32.6	32.3
KANSAS CITY, MO	366.1	706.4	32.0	48.9	49.2
WAVERLY, MO	293.4	646.0	20.0	31.2	31.5
BOONVILLE, MO	197.1	565.4	21.0	37.1	36.9
HERMANN, MO	97.9	481.6	21.0	36.2	36.1

TABLE KC-2
MISSOURI RIVER
DIFFERENCES IN COMPUTED PEAK STAGES
ALTERNATIVE SIMULATIONS VS. 1993 FLOOD COMPUTED BASE CONDITIONS

MISSOURI RIVER GAUGE	RIVER MILE	AGRICULTURAL LEVEES REMOVED		1993 FLOOD TOTALLY CONFINED BY LEVEES	25-YEAR LEVEES
		AGRICULTURAL OVERBANKS - LOW ROUGHNESS	NATURAL OVERBANKS - HIGH ROUGHNESS		
ST. JOSEPH	448.2	-3.0	-2.9	+1.6	-5.0
KANSAS CITY	366.1	-1.2	-2.9	+2.8	-4.5
WAVERLY	293.4	-2.7	-0.7	+6.9	-0.7
BOONVILLE	197.1	-0.1	+1.8	+4.1	-0.3
HERMANN	97.9	+1.0	+4.6	+6.8	-0.8

MISSOURI RIVER GAUGE	RIVER MILE	NO FEDERAL RESERVOIRS	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%	AGRICULTURAL LEVEE SETBACKS
ST. JOSEPH	448.2	+0.4	-0.2	-0.6	+0.4
KANSAS CITY	366.1	+5.1	-1.1	-2.2	-0.5
WAVERLY	293.4	+1.2	-0.3	+1.2	+0.7
BOONVILLE	197.1	+1.4	-0.5	-0.4	+1.0
HERMANN	97.9	+3.6	-0.2	+0.6	+0.6

NOTE: Stage differences are listed in units of feet.

TABLE KC-3
MISSOURI RIVER
PERCENT CHANGE IN MAXIMUM DISCHARGE
ALTERNATIVE SIMULATIONS VS. 1993 FLOOD COMPUTED BASE CONDITIONS

MISSOURI RIVER GAUGE	RIVER MILE	AGRICULTURAL LEVEES REMOVED		1993 FLOOD TOTALLY CONFINED BY LEVEES	25-YEAR LEVEES
		AGRICULTURAL OVERBANKS - LOW ROUGHNESS	NATURAL OVERBANKS - HIGH ROUGHNESS		
ST. JOSEPH	448.2	-5.3%	-22.0%	+15.1%	-35.5%
KANSAS CITY	366.1	-4.1%	-14.2%	+9.4%	-19.0%
WAVERLY	293.4	-3.0%	-11.5%	+8.7%	-1.3%
BOONVILLE	197.1	+2.9%	-4.6%	+12.1%	-13.6%
HERMANN	97.9	+3.5%	-4.7%	+13.5%	-12.1%

MISSOURI RIVER GAUGE	RIVER MILE	NO FEDERAL RESERVOIRS	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%	AGRICULTURAL LEVEE SETBACKS
ST. JOSEPH	448.2	+4.2%	-2.5%	-5.8%	+3.9%
KANSAS CITY	366.1	+23.5%	-3.8%	-8.3%	-2.5%
WAVERLY	293.4	+14.4%	-4.5%	-5.9%	-2.6%
BOONVILLE	197.1	+15.9%	-5.7%	-4.4%	+0.4%
HERMANN	97.9	+31.3%	-5.4%	-5.6%	+0.3%

TABLE KC-4
MISSOURI RIVER
LEVEE PARAMETERS

LEVEE SYSTEM NAME	LEFT OR RIGHT BANK	UPSTREAM RIVER MILE	DOWNSTREAM RIVER MILE	PROTECTED AREA (Acres)
Duke-Mills	L	498.1	492	3870
R-513	R	497.5	495	1443
Fortescue	L	492	486.5	10520
Osgood	R	491.8	489.8	655
Windle	L	486.3	482.8	10150
R-500	R	484.5	480	1430
L-497	L	482.8	476.2	6480
Earle Cole	R	478	474.7	922
L-488	L	476	465.2	8470
R-482	R	468	458	4650
L-476	L	460.7	454	5565
R-471-460	R	456.5	441.8	13035
L-455	L	445.6	437.5	6922
L-448-443	L	437.5	428	14550
R-440	R	431	424.3	4352
Rushville D. D.	L	428	418.2	9389
Kemig	R	418.7	416.7	598
Ohlhausen-Johnson	L	418	403.5	12526
Hundley	R	416.7	415	768
Pohl	R	412	406.5	2163
Sherman Field	R	403	399.3	1448
L-408	L	401.3	391.5	9660
Ramsey	R	394	388.2	3065
L-400	L	391	385	3798
Barcus	R	386.2	381.2	2220
Vandiver-Oldham	L	383	379	2673
Quindaro Bend L. D.	L	376	372	1594
Fairfax Jersey Creek	R	374	367.5	2080
North Kansas City	L	370.5	363.5	2924

TABLE KC-4 (CONTINUED)
MISSOURI RIVER
LEVEE PARAMETERS

LEVEE SYSTEM NAME	LEFT OR RIGHT BANK	UPSTREAM RIVER MILE	DOWNSTREAM RIVER MILE	PROTECTED AREA (Acres)
C.I.D.	R	367.4	365.7	860
East Bottoms	R	365.7	357.5	4961
Birmingham	L	360.3	353.5	5050
Courtney D. D.	R	354	352	597
Liberty Bend Levee	L	353	352	719
Allen	L	351.8	348.3	3970
R-351	R	350	339.7	7957
Bell	L	348.3	345.6	1164
Jarvis-Rone Allen Pigg	L	341.8	338	6215
Egypt Levee District	L	338	334.3	4700
Yates	L	333.4	326.2	11535
Lafayette Co. L. D. #1	L	326.2	324	7218
Sunshine L. D.	L	324	317	11130
Ray Co. L. D. #6	L	317	313.8	16100
New Crooked River D. D.	L	313.8	312.2	2660
Fredendall	R	312.5	310	1375
Ray & Carroll L. D. #2	L	312	268.2	121000
Hodge Bottoms	R	300.5	296	1044
Saline-Lafayette D. D.	R	292.5	278.3	10139
Malta Bend L. D.	R	278.3	273.6	5140
Teteseau Bend L. D.	R	273.5	263.2	11047
Ray Fail	L	268.2	263	2394
Mi-De L.D.	L	263	257	5150
Miami-Dewitt Bend	R	261.8	251.8	3409
Dewitt Levee D. D.	L	257	255.2	2809
Brunswick L. D. #1	L	255.2	250.5	3803
Stonner	R	252	245.5	3890
L-246	L	250	239	28413
Chariton River Mainstem	L	238.8	227.3	20160

TABLE KC-4 (CONTINUED)
MISSOURI RIVER
LEVEE PARAMETERS

LEVEE SYSTEM NAME	LEFT OR RIGHT BANK	UPSTREAM RIVER MILE	DOWNSTREAM RIVER MILE	PROTECTED AREA (Acres)
Grossermeyer	R	235	231.5	1283
Noth	R	231	217.6	8670
Carmack	L	222.5	220	458
Denny, et al	L	216	213.5	612
Thompson, et al	R	215.5	210.7	1997
Schnuck	L	212	204	5809
Linneman	R	206.5	203.5	1043
Howard Co. D. D. #1	L	204	198	8685
Franklin Island	L	198	192.5	5575
Ambrose	L	192	190	4617
Farris	R	191.2	188	900
Stegner	L	190	187.2	808
Overton-Wooldridge (1)	R	188	183	3904
Overton-Wooldridge (2)	R	182.5	177.5	2805
McBaine	L	180	175.5	2567
Holman	L	175.5	171	1855
Plow Boy Bend	R	172.7	166.4	2576
Easley-Wilton	L	168.5	162	2664
Capital D. D.	R	163.8	158.2	2500
Hartsburg L. D. (1)	L	160.5	155.3	2223
Hartsburg L. D. (2)	L	155.3	153.5	1274
Hartsburg L. D. (3)	L	153.5	150.8	875
Cole Junction L. D.	R	154.7	151.5	1148
Prison Farm	R	151.5	146	2721
Jose	L	148.2	144.3	1816
Capitol View L. D.	L	144.3	140	3657
Cedar Island	L	140	136.7	2584
Wainwright D. D.	L	136	134	1831
Osage River Levee	R	135	132.5	525

TABLE KC-4 (CONTINUED)
MISSOURI RIVER
LEVEE PARAMETERS

LEVEE SYSTEM NAME	LEFT OR RIGHT BANK	UPSTREAM RIVER MILE	DOWNSTREAM RIVER MILE	PROTECTED AREA (Acres)
Stock	L	130.3	125	3560
MoKane (1)	L	124.8	123.2	1390
MoKane (2)	L	123	120.5	943
Chamois	R	122.2	118.7	591
Steedman	L	120.5	115.3	2925
Stoner Island	L	118.4	115.8	817
Rhodes-Means	R	117	110	5083
Tri-County L. D.	L	109	98.2	8028
Lieneke	R	108	106.5	1010

TABLE KC-5
MISSOURI RIVER
LEVEE PERFORMANCE
1993 FLOOD EVENT AND ALTERNATIVE SIMULATIONS

MISSOURI RIVER LEVEE SYSTEM	1993 FLOOD	LEVEES REMOVED-AGRICULTURAL	LEVEES REMOVED-NATURAL	CONFINED BY LEVEES	25-YEAR LEVEES	NO FEDERAL RESERVOIRS	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%	LEVEE SETBACKS
DUKE-MILLS	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
R-513	NO	REMOVED	REMOVED	NO	NO	NO	YES	YES	NO
FORTESCUE	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
OSGOOD	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
WINDLE	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
R-500	YES	REMOVED	REMOVED	NO	YES	NO	YES	YES	NO
L-497	NO	REMOVED	REMOVED	NO	YES	NO	YES	YES	NO
EARLE COLE	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
L-488	YES	REMOVED	REMOVED	NO	YES	NO	YES	YES	NO
R-482	YES	REMOVED	REMOVED	NO	YES	NO	YES	YES	NO
L-476	NO	REMOVED	REMOVED	NO	YES	NO	YES	YES	NO
R-471-460	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	NO
L-455	NO	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
L-448-443	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
R-440	NO	REMOVED	REMOVED	NO	YES	YES	YES	NO	YES
RUSHVILLE D.D.	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
KEMIG	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
OHLHAUSEN-JOHNSON	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
HUNDLEY	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	----
POHL	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
SHERMAN FIELD	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES

YES: MEANS THE LEVEE WAS OVERTOPPED NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE KC-5 (CONTINUED)
MISSOURI RIVER
LEVEE PERFORMANCE
1993 FLOOD EVENT AND ALTERNATIVE SIMULATIONS

MISSOURI RIVER LEVEE SYSTEM	1993 FLOOD	LEVEES REMOVED-AGRICULTURAL	LEVEES REMOVED-NATURAL	CONFINED BY LEVEES	25-YEAR LEVEES	NO FEDERAL RESERVOIRS	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%	LEVEE SETBACKS
L-408	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
RAMSEY	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
L-400	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
BARCUS	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
VANDIVER-OLDHAM	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
QUINDARO BEND LD	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
FAIRFAX JERSEY CK	NO	NO	NO	NO	NO	YES	NO	NO	NO
NORTH KANSAS CITY	NO	NO	NO	NO	NO	YES	NO	NO	NO
C.I.D.	NO	NO	NO	NO	NO	YES	NO	NO	NO
EAST BOTTOMS	NO	NO	NO	NO	NO	YES	NO	NO	NO
BIRMINGHAM	NO	NO	NO	NO	NO	NO	NO	NO	NO
COURTNEY D.D.	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
LIBERTY BEND	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
ALLEN	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
R-351	NO	REMOVED	REMOVED	NO	YES	YES	YES	NO	YES
BELL	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
JARVIS-RONE A.P.	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
EGYPT LEVEE DIST.	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
YATES	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
LAFAYETTE CO LD#1	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
SUNSHINE L.D.	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES

YES: MEANS THE LEVEE WAS OVERTOPPED NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE KC-5 (CONTINUED)
MISSOURI RIVER
LEVEE PERFORMANCE
1993 FLOOD EVENT AND ALTERNATIVE SIMULATIONS

MISSOURI RIVER LEVEE SYSTEM	1993 FLOOD	LEVEES REMOVED-AGRICULTURAL	LEVEES REMOVED-NATURAL	CONFINED BY LEVEES	25-YEAR LEVEES	NO FEDERAL RESERVOIRS	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%	LEVEE SETBACKS
RAY CO. L.D. #6	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
NEW CROOKED R. DD	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
FREDENDALL	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
RAY & CARROLL #2	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
HODGE BOTTOMS	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
SALINE-LAFAYETTE	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
MALTA BEND LD	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
TETESEAU BEND LD	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
RAY FAIL	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
MI-DE L.D.	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
MIAMI-DEWITT BEND	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
DEWITT LEVEE DD	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	----
BRUNSWICK L.D.#1	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
STONNER	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
L-246	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
CHARITON MAINSTEM	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES	YES
GROSSERMEYER	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
NOTH	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
CARMACK	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
DENNY, ET AL	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
THOMPSON, ET AL	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES

YES: MEANS THE LEVEE WAS OVERTOPPED NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE KC-5 (CONTINUED)
MISSOURI RIVER
LEVEE PERFORMANCE
1993 FLOOD EVENT AND ALTERNATIVE SIMULATIONS

MISSOURI RIVER LEVEE SYSTEM	1993 FLOOD	LEVEES REMOVED-AGRICULTURAL	LEVEES REMOVED-NATURAL	CONFINED BY LEVEES	25-YEAR LEVEES	NO FEDERAL RESERVOIRS	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%	LEVEE SETBACKS
SCHNUCK	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
LINNEMAN	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
HOWARD CO. DD #1	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
FRANKLIN ISLAND	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
AMBROSE	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
FARRIS	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
STEGNER	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
OVERTON-WOOLDRG.1	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
OVERTON-WOOLDRG.2	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
MCBAINE	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
HOLMAN	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
PLOW BOY BEND	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
EASLEY-WILTON	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
CAPITAL D.D.	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
HARTSBURG L.D.1	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
HARTSBURG L.D.2	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
HARTSBURG L.D.3	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
COLE JUNCTION LD	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
PRISON FARM	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
JOSE	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
CAPITOL VIEW L.D.	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES

YES: MEANS THE LEVEE WAS OVERTOPPED NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE KC-5 (CONTINUED)
MISSOURI RIVER
LEVEE PERFORMANCE
1993 FLOOD EVENT AND ALTERNATIVE SIMULATIONS

MISSOURI RIVER LEVEE SYSTEM	1993 FLOOD	LEVEES REMOVED-AGRICULTURAL	LEVEES REMOVED-NATURAL	CONFINED BY LEVEES	25-YEAR LEVEES	NO FEDERAL RESERVOIRS	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%	LEVEE SETBACKS
CEDAR ISLAND	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
WAINWRIGHT D.D.	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
OSAGE RIVER LEVEE	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
STOCK	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
MOKANE 1	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----
MOKANE 2	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
CHAMOIS	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
STEEDMAN	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
STONER ISLAND	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
RHODES-MEANS	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
TRI-COUNTY L.D.	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	YES
LIENEKE	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES	----

YES: MEANS THE LEVEE WAS OVERTOPPED NO: MEANS THE LEVEE WAS NOT OVERTOPPED

FPMA

FloodPlain Management Assessment

Hydrology and Hydraulics

St Paul District

May 1995
Final Report

US Army Corps
of Engineers

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Description of the Mississippi and Missouri River Basins

Watershed Characteristics

The Mississippi River rises in the lake and forest country of north-central Minnesota, near Itasca, Minn., and flows north, east, and then south through timbered landscape to Minneapolis-St. Paul. At this point, it leaves the northern woodlands and lakes and meanders southward past fertile prairies and many villages and cities. Along the way, tributaries that drain lands to the east and to the west join the Mississippi River and add to its flow. The Mississippi River basin drains 41 percent of the land area of the continental United States and covers all or part of 31 states. Starting with its headwaters in the upper Minnesota Lake Itasca region, the river flows 2,350 miles to its mouth in the Gulf ofMexico. The flood plain along the main stem of the Mississippi River varies from approximately three quarters ofa mile to more than 14 miles wide and averages about five miles wide. The river, which originally followed a meandering course, has been fixed with much of its adjacent farm land now protected by levees.

The drainage area of the Mississippi River is comprised of six major sub-basins: the Ohio, upper Mississippi, Missouri, Arkansas, White, and the lower Mississippi. Each sub-basin contributes flow to the main stem Mississippi River in varying amounts. Historically, the Missouri and Arkansas rivers have contributed greater concentrations of sediment, while the Ohio River contributes the greater percentage of water discharge and the least concentration of sediment. The total drainage area of the Mississippi River is approximately 717,600 square miles at its confluence with the Ohio. Approximately 523,000 square miles of this area represents the Missouri River basin. Plate SP-1 shows a map of the basin and the area covered by this floodplain assessment. The Missouri River drains approximately 9,700 square miles of Canada and 513,300 square miles or one sixth of the contiguous United States. Its area includes all of Nebraska and parts of Missouri, North Dakota, Kansas, Colorado, Wyoming, South Dakota, Iowa, and Minnesota. The watershed drained by the Missouri River flows 2,315 miles from Three Forks, Mt. to join the Mississippi River near St. Louis, Mo. Other principle tributaries of the Mississippi River include the Salt, Illinois, Kaskaskia, Meramec, Big Muddy, St. Francis, Rock, Des Moines, Iowa, Wisconsin, St. Croix, and Minnesota rivers.

Antecedent Conditions

Antecedent conditions had a major influence on the runoff that was experienced in 1993. Precipitation in the upper Mississippi River basin in the fall of 1992, November and December were well above normal. Although not record breaking, the snow cover in the upper Mississippi River Basin at the beginning of the 1993 spring season was somewhat greater than normal, particularly in southern areas. Across southern Minnesota and western and central Wisconsin, snow depths at the end of February 1993 were generally in the 9- to 18-inch range with water equivalents in the 2- to 4- inch range. Frost penetration ranged from 14 inches at Lamberston to 34 inches at Morris in Minnesota, with similar range in western and central Wisconsin. These values are not abnormal, and suggest that snow and soil conditions at the end of winter on 1992-93 were not significant contributing factor to the floods of the summer 1993. Melting snow did, however, combine with above normal spring rains and below normal spring temperatures to adversely impact soil moisture conditions and deplete much of the depressional storage in the watershed.

Precipitation during the winter of 1992-1993 and spring of 1993 was above normal and temperatures were below normal throughout the lower Missouri River basin. Persistent rains and early snowmelt culminated in high spring runoff. With the exception of some areas in Colorado and western Kansas, which had below normal precipitation, the period of April and May was wet and cool.

The situation began to set up for the summer months in April and particularly May. Precipitation in April was about twice the normal in the southern half of Wisconsin and the northern two-thirds of Illinois. Five inches or more of rain fell over a wide area, including southeastern Minnesota, the southern half of Wisconsin and most of Illinois. In May, heavy rainfall occurred in Iowa and Missouri with a monthly total of 8 inches; 4-6 inches of rain fell in Minnesota and Wisconsin; and 6 inches fell in Illinois. The above normal precipitation fell in areas most directly impacted by the heavy rains to come. This was particularly the case in southwestern Minnesota, where two storms in succession between May 6 and 8 brought the first of the damaging floods and left the soil saturated, setting the stage for the more widespread flooding a month later.

There are a number of conditions which can affect runoff in a river basin and result in major flooding. The four most significant such conditions relevant to the floods of the summer of 1993 in the Upper Mississippi and Lower Missouri river basins were base flow, snow cover, soil moisture, and antecedent rainfall. Base flows were well above normal on most of the streams in the affected area. Snow cover was not excessive, but when combined with the Spring rainfall it produced moderate flooding in some areas. Soil moisture across Minnesota, Wisconsin and Iowa in the spring of 1993 was extremely high, making this a significant contributing factor to the floods of the summer of 1993. The following shows soil moisture as a percent of capacity:

Minnesota	85 %
Iowa	85 %
Wisconsin	75 %
Illinois	80 %

These high values meant that a large percentage of new precipitation had nowhere to go but directly into runoff. This above normal precipitation was accompanied by significantly below normal temperatures. Mean April temperature ranged from 3 to 4 degrees below normal across the entire area, with isolated stations reporting monthly averages about 7 degrees below normal. Monthly average temperature for May were colder than normal by 1.5 to 2.5 degrees. This combination of precipitation and temperature had several effects. The above normal precipitation, combined with the melted winter snow pack, left soils very close to saturation. The cooler temperatures inhibited evapotranspiration, furtherpromoting saturated soil conditions and ponding in fields. Both of these conditions delayed planting and inhibited crop rootgrowth, which further contributed to excessive runoff.

Description of Storms

One of the unusual aspects of the floods of 1993 was that they were not the product of one single, large-scale event, such as an intense synoptic scale cyclone or snowmelt and runoff. Rather, they were the result of numerous smaller scale and shorter duration, but more locally intense, thunderstorm events which were much more widespread and longer lasting than individual events of this kind usually are.

The flood-producing rainfall events were typically the result ofthunderstorms repeatedly forming and moving over the same area, a phenomenon sometimes referred to as the "train effect." Storms of this kind usually form right along, or just to the north or northwest of a slow moving or stationary front aligned parallel or nearly parallel to the upper air winds. Weather disturbances moving alongthe surface front will cause the warmer air to the south or southeast of the front to be forced to rise over the cooler air to the north or northwest. In an area determined by the air mass and circulation characteristics, the warm air will have risen

to a level where it will begin to rise freely and rapidly due to convection, generating thunderstorms which then move with the upper winds. In these situations, it is common for thunderstorms to form in and then move over the same areas, one after the other, creating the "train effect." The alignment of the surface fronts and the jet stream during the summer of 1993 were highly favorable for the formation of the kind of weather disturbances which set off the "train effect" thunderstorms. The intensity of these storms, once they formed, was then enhanced by the extreme nature of the temperature contrasts across the region and the intensity of the jet stream.

By the summer of 1993, the mean position of the jet stream was firmly established over the northern portion of the Mississippi River basin with a southwest-northeast orientation. Major flooding began after a particularly heavy rainfall period in mid-June in southwest Minnesota and northwest Iowa. This included record flooding on the Minnesota River. Following a short dry period, the area experienced a prolonged siege of heavy rainfall from late June extending through July 11. This included extreme precipitation on July 9 in Iowa, which resulted in record flooding on the Raccoon and Des Moines rivers. Just as the crests from these two rivers reached Des Moines, a relatively small, convective pocket dumped several inches of rain on the crests, rapidly boosting the river levels and flooding a water treatment plant in Des Moines. This rainfall event also led to record flooding on portions of the lower Missouri River and combined with the crest already moving down the Mississippi, causing record river stages from the Quad Cities area, through St. Louis, and as far south as Thebes, Ill. Another major precipitation impulse occurred July 21 to 25. The heaviest rains were focused farther south than the earlier events, with especially heavy rain falling over eastern Nebraska and Kansas, leading to second major crests on both the Missouri and Mississippi rivers.

Chronology of Storms

The following is a chronology of some of the more notable storms that occurred over the region from June to August. In June and July, rain fell somewhere in the region every day.

June 16-18— 2-7 inches of rain fell throughout southern Minnesota, northern Iowa, and southwestern Wisconsin, areas with already saturated soils. The heaviest rain fell directly over the Minnesota River. These storms caused flooding on the Minnesota and Mississippi rivers in Minnesota and the Chippewa and Black rivers in Wisconsin which began the entire Mississippi River flood event. Further precipitation in the next few days caused flooding in and near Black River Falls, Wis. It also caused flooding in other tributary basins in Wisconsin: the Chippewa, Buffalo, Trempealeau, and Wisconsin basins.

June 25—Additional localized rainfall in central Iowa contributed to the runoff at the three Iowa reservoirs—Saylorville, Coralville, and Red Rock.

June 27—Several areas recorded up to 4 inches of rainfall. The Iowa River basin below Coralville Lake was one of the areas that received heavy precipitation. The Papillion Creek basin in Omaha, Neb. experienced 3-5 inches of rain.

June 28—Additional rainfall around Iowa City, Iowa, and the upper Mississippi River below Dubuque, Iowa, continued to aggravate the situation.

June 29—An additional 2 inches of rain fell on Iowa City and the upper Mississippi River during the night. Seven inches of rain occurred over the Lake Okoboji and Spirit Lake area in Iowa.

July 1—Near Quincy, Ill., an additional 2-5 inches of precipitation fell. Flood waters continued to rise along a 300-mile stretch of the Mississippi River on July 2. On July 6, the Mississippi River crested for the second time at Dubuque. The second crest continued downstream to the Quad-Cities, Keithsburg, Ill., and Hannibal, Mo., and new records were established.

July 2-5— 5-7 inches of rain was reported in an area from Mitchell to Madison, S.D.

July 3-9—Six to ten inches of rain fell in various locations in Iowa, Kansas, and Missouri. Rain on July 3 caused the third episode of significant flooding in Marshall, Minn., in two months.

July 4-5—This storm was a significant event that produced a large amount of rainfall over southern Iowa. It produced a total of 4-8 inches of rain across a 250-mile long path from Taylor County in southwest Iowa, northeastward through Oskaloosa, Marengo, Cedar Rapids, and Dubuque.

July 7-8—Additional rainfall occurred on the Des Moines River basin. The rivers throughout central Iowa had not receded from the July 4 storm and the three major reservoirs in the area were at capacity. Strong thunderstorms moved into central Iowa before sunrise on July 8 and rapidly traversed eastward across Iowa and into Illinois. A second set of thunderstorms developed over west-central Iowa later in the afternoon and slowly moved along the same path as the morning storms. By the time these storms weakened on July 9, a wide area of 3-9 inches of rain fell in an uninterrupted 275-mile long band from the Nebraska border at Onawam eastward through Guttenburg, Iowa. Up to 8.5 inches of rain fell on the Des Moines River basin at Jefferson, Iowa. Marshalltown, Iowa, received up to 3 inches while 1-2 inches of rain fell over various parts of eastern Iowa and western Illinois. The Mississippi River crested for a third time at Camanche, Iowa, and the Quad-Cities also crested again. The runoff from the July 4 and July 8 storms caused record or near-record peak discharges on the Iowa, Skunk, Raccoon, and Des Moines river basins. The flood peaks from these tributaries entered the Mississippi River at about the same time the flood peak from the late June storm in the northern basins reached Keokuk, Iowa. The crest approached St. Louis from the north and joined high water coming in from the west down the Missouri River, an event which has never occurred since the Corps of Engineers has been keeping records.

July 11—Moderate to heavy rainfall fell in central Illinois.

July 13—Heavy rainfall occurred in the city of Des Moines area.

July 15-16—Up to seven inches of rain fell in eastern North Dakota and western Minnesota. These storms caused flooding in the upper reaches of the Minnesota River basin in Minnesota and the James River basin in North Dakota.

July 17—Two to five inches of rain was reported in about a one-hour period over the Mill Creek basin near Cherokee, Iowa.

July 18—Light to heavy precipitation occurred across Iowa, Wisconsin, and Illinois. Portions of the Cedar River basin received up to an additional five inches of rainfall. Heavy rains caused flooding on the Baraboo River in Wisconsin.

July 22-25—Up to 13 inches of rain fell in parts of Nebraska, Kansas, North Dakota, Missouri, Iowa, and Illinois, resulting in peak stages along the Missouri River south of Omaha, Neb. On July 24, an

additional four inches of rain fell on southern portions of Iowa and Illinois. The Mississippi River began to climb again and the Illinois Waterway also went above flood stage. There were unofficial reports of up to 16 inches of rainfall in southeast Nebraska.

July 31—Significant precipitation occurred in eastern Iowa. Iowa City and areas south reported two to three inches of additional rain.

Aug. 10—Up to four inches of rain fell near Iowa City.

Aug. 11—Additional precipitation occurred throughout the area with up to five inches falling in the Iowa River and Cedar River basins. Flash flooding occurred along the Iowa River near Marshalltown and Tama, Iowa, in the same area that experienced flooding previously.

Aug. 20—Six inches of rain occurred in two hours over the southern Black Hills of South Dakota.

Aug. 21—Seven to 10 inches of rain fell near Wolf Point, Mont.

Description of Flooding

The Great Flood of 1993 was unique in its areal extent as well as in its duration. It encompassed several months of relatively heavy rainfall that occurred at a time when the ambient conditions already posed a greater probability for flooding. Along the Mississippi River, many of the federal and non-federal levees either overtopped or were breached as a result of the record-breaking stages. The flooding on the Mississippi River was the most devastating in terms of property, disrupted businesses, and personal trauma of any in the history of the United States. Millions of acres of farmland were under water for weeks during the growing season. Damaged highways and roads disrupted overland transportation throughout the flooded region. The river was closed to navigation for several weeks. The banks and channels of the Mississippi River were severely eroded in many reaches. In addition to the erosion of the river, erosion of valuable topsoil was a major problem. The extent and duration of the flooding caused numerous levees to fail. The flood affected a large portion of the midwestern United States, crossing boundaries of several Corps of Engineers districts, including: St. Paul, Rock Island, Omaha, Kansas City, and St. Louis. Each of these districts experienced some degree of flooding during the spring and summer of 1993.

Flood effects along the main stem of the Mississippi River were generally confined to near-bank areas and channel infrastructure from St. Paul, Minn., to Guttenberg, Iowa. There was no significant flooding upstream of Lock and Dam No. 1 in Minneapolis, Minn. Every gaging station on the Mississippi River below Lock and Dam No. 15 to Thebes, Ill., experienced a new flood of record. Above Lock and Dam 15, the 1993 flood was surpassed by only one other event. Flood conditions on the Mississippi River differed above and below the confluence of the Ohio River. At Thebes, Ill., 46 miles upstream from the confluence, severe flooding occurred on the Mississippi. Downstream from the confluence, flooding on the Mississippi River was not severe because of less-than-average discharge contributed by the Ohio River and a substantially larger channel capacity in this reach of the Mississippi River. The discharge of the Ohio River was less than average during July and August as a result of generally dry conditions and low reservoir outflows throughout the Ohio River.

The wet spring of 1993 resulted in the Missouri River rising above flood stage in early May and navigation being suspended from river mile 197.0 to 354.0. By May 16, the river was reopened to navigation

and the flood event was terminated on May 20. This relatively minor event set the stage for a series of events that would result in record flows and stages on the Missouri River and record pool levels at several lake projects during the months of July and August. Hydrologic and hydraulic effects of excessive runoff during the summer of 1993 resulted in severe and widespread flooding throughout the lower Missouri River basin in Missouri, central and east Kansas, southeast Nebraska and south central and southwest Iowa. Several intense storms in July, combined with wet antecedent conditions were the principle causes of the severe flooding conditions. Record flooding inundated large areas—residential, industrial, and agricultural. The extent and duration of flooding caused levees on the Missouri River to fail or be overtopped. The Missouri River was closed to navigation for 49 days, from July 2 to Aug. 20.
Even after the record setting flood had passed out of the Missouri basin, during Augustand September, there continued to be rainfall that caused recurrences of flooding in localized areas.

Existing Land Use

Existing land use and its impacts on runoff in the basin is guided by Federal, state, and local programs, regulations, and law. The measures covered under these programs include both structural and non-structural measures to reduce flooding. These measures are implemented both in the floodplain to directly reduce flood impacts and in the uplands to reduce the runoff volumes and flood peaks. The structural measures include an extensive system of levees, floodways, channel maintenance, reservoirs, and tributary improvements. These structures exist because the public has wanted them and were built as a result of petitions of local residents to their Congressional representatives for relief from flooding. The upland measures include flood control reservoirs (structural) and USDA supported programs (non-structural) which directly affect the runoff from the land.

Tables SP-1 and SP-2 provide summaries of various land use categories as derived from the NRCS 1992 Natural Resource Inventory (NRI) and the State Soil Geographic Database (STATSGO) of the NRCS. The data is an extension of the inventory summarized in the "Summary Report, 1992 National Resources Inventory" by the USDA and the NRCS. The data is supplied through the cooperation of the NRCS Midwest National Technical Center, Lincoln, Nebraska.

TABLE SP-1
UPPER MISSISSIPPI RIVER BASIN
LAND USE INVENTORY

BASIN NAME	BROAD LAND USE CATEGORIES (1000 ACRES)											
	HUC#	TOTAL AREA	WATER	CROP CULTIV.	CROP NON-CULT	FEDERAL LANDS	FOREST	MISC.	PASTURE	RANGE	RURAL TRANS	SML/LRG URBAN
MISS R. HEADWATERS	701	13098	1189	2587.8	1001.2	518.5	4175.1	1687.8	1078.9	0	276.6	584
MINNESOTA RIVER	702	10840	338	7328.9	309.5	90.3	277	965.8	692	261.9	346.2	229.9
ST. CROIX RIVER	703	5012	214	535.8	426.4	97.5	2374.4	597.3	529.7	0	88.7	149.3
ZUMBO/ROOT R'S	704	6911	122	2616.6	657.1	139.8	1724.1	623.8	669.3	0	173.6	185.3
CHIPPEWA R.	705	6183	276	736.4	639.2	462.6	2763.9	569.9	515.5	0	113.8	106.5
MAQUOKETA/PLUM R	706	5386	142	2357.8	494.5	79.4	853.9	475.1	733.5	0	149	101.6
WISCONSIN R.	707	7616	282	1147	787.1	271.6	3364.7	654.5	733.3	0	159.5	217
IOWA/SKUNK R'S	708	14694	224	10211.1	516.6	71.7	757.5	1071.6	1022.8	0	404.6	413.8
ROCK R.	709	7091	147	4074.6	585.8	31.7	449.4	565	569	0	193.8	474.7
DEMOINES R	710	9195	135	6081.8	358.6	64.4	506.3	630	972.7	0	257.6	188.9
WYACONDA/SALT	711	6462	172	2941.4	334	72.5	1046.9	556.8	1037.6	0	141.2	160.3
UPPER ILL.	712	6956	145	4038.1	192.8	34.5	339.2	291.8	349.1	0	163.1	1402.7
LOWER ILL.	713	11494	217	8281	230.3	29.4	954.6	268.1	920.1	0	246.8	347
KASKASKIA/MERAMAC	714	10874	280	4008.6	441.8	485.4	2806.7	443.5	1559.3	0	216.1	633.5
WHITE R	1014	12966	286	2617.2	662.2	683.3	273.9	829.2	436.4	6988.	164.6	24.6
NIOBRARA R	1015	8981	71	1109.6	543.6	221.3	149.4	277.1	244.8	6248.	99.6	16.1
JAMES R	1016	14547	229	7232.9	854.7	187.3	38.4	1464.5	753.1	3320.	391.8	74.7
BIG SOIUX	1017	9172	217	5618.3	532.6	67.8	75.2	754.9	740.6	786.5	258	120.5
NORTH PLATTE	1018	20494	226	1024.5	547.7	5376.9	223.6	813.8	355.9	11725	140.2	60.4
SOUTH PLATTE	1019	15214	147	3830.8	346.7	2097.7	726.3	876.4	351.4	5922.	263.8	651.5
LOWER PLATTE	1020	5291	151	2628	235.3	29.5	110.9	201.5	282.7	1410.	115.9	127
LOUP R	1021	9772	135	1358.1	348.3	137.1	26.3	188.7	299.1	7168.	91.3	20.3
ELKHORN R	1022	4403	30	2194.7	230.6	3	45.2	279.9	285.3	1212.	87.1	34.8
LITTLE SIOUX	1023	6038	98	4273.7	131.2	26.7	163.6	427	510.3	0	179.8	227.6
NISHNABOTNA	1024	8646	125	5176.1	346.2	31.2	482.8	690.6	1367.2	31.5	217.2	179.1
REPUBLICAN R	1025	16065	80	7957.2	219.3	159.5	74	941.3	161.6	6127.	290.3	54
SMOKEY HILL R	1026	12725	101	6489.1	239	110.9	122.1	819.6	59.3	4453.	245.9	85.7
KANSAS R	1027	9729	137	5081	414.7	175.6	382.6	415	855.9	1789.	245.8	233
CHARITON/GRAND R	1028	7042	100	2152.7	643.3	34.2	690.2	1029.1	2128.7	0	185.3	78
GASCONADA/OSAGE R	1029	11858	300	1707.1	773.4	429.4	3124	542.6	3890.2	620.6	242.8	228
LOWER KANSAS	1030	6619	142	1957.3	516.6	42.9	1350.5	329.4	1674	0	141.4	464.2
TOTALS		301390	6465	119355	14560	12264	30453	20282	25779	58067	6291	7874

(1) Data derived from NRCS 1992 NRI.

TABLE SP-2

UPPER MISSISSIPPI RIVER BASIN
ESTIMATED WETLAND ACREAGES BY HYDROLOGIC UNIT

BASIN NAME	HYDRO UNIT HUC#	D.A. 1000 ACRES	D.A. SQUARE MILES	HYDRIC SOILS 1000 ACRES (1)	HYDRIC SOILS PERCENT OF TOTAL AREA	COWARDIN WETLAND CLASSIFICATIONS (2) (1000 ACRES)							
						RIVERINE		LACUSTRINE		PALUSTINE			
						OTHER	EMERG	OTHER	EMERG	OTHER	EMERG	SCR-SH	FOREST
MISS R. HEADWATERS	701	13098.	20467.	3897	29.75%	38.9	14.8	1109	25.8	243.3	892.7	649.2	964.4
MINNESOTA RIVER	702	10840.	16938.	4318	39.83%	34.3	16.4	264.9	31.3	783.5	450.6	21.2	58.7
ST. CROIX RIVER	703	5012.	7832.	1285	25.63%	19.9	0	168.6	22.3	20.3	403.2	236.3	368.1
ZUMBO/ROOT R.	704	6911.	10799.	1096	15.86%	41.2	0.9	60	15.3	47	135.4	92.9	222.2
CHIPPEWA RIVER	705	6183.	9661.	1164	18.82%	49.7	0	219.5	4.7	23.4	193.4	344.3	555.1
MAQUOKETA/PLUM R.	706	5386.	8416.	367	6.81%	95.2	4	37.9	1.6	18.2	67.4	5.4	26.7
WISCONSIN RIVER	707	7616.	11901.	1328	17.43%	51.1	1.7	212.7	4.3	79.9	381.1	206.1	627.1
IOWA/SKUNK R'S	708	14694.	22960.	3683	25.06%	165.1	5.3	32.4	0	41.7	312.7	5.6	197.7
ROCK R.	709	7091.	11080.	1223	17.25%	54.5	3.6	65.3	11.2	130	340	15.7	67.9
DEMOINES RIVER	710	9195.	14367.	3339	36.31%	66.5	0.9	47.7	4.5	109.8	257.3	0	58.5
WYACONDA/SALT	711	6462.	10097.	1518	23.49%	104	3.8	32.9	0	55.4	47.3	7	131.5
UPPER ILL.	712	6956.	10868.	2212	31.80%	58	0	61.3	13.2	45	151.7	13.7	81.2
LOWER ILL.	713	11494.	17960.	2779	24.18%	96.9	2.9	73.1	20.8	123.7	33.5	2.9	124.6
KASKASKIA/MERAMAC	714	10874.	16991.	2382	21.90%	122.8	11.4	84.6	9.7	212.4	17.5	15.9	155.1
WHITE RIVER	1014	12966.	20259.	324	2.50%	41.8	0	214.9	1	63.2	233.8	9.1	0
NIOBRARA RIVER	1015	8981.	14033.	378	4.21%	44.5	3	11	0	51.3	231.9	0.8	7.8
JAMES RIVER	1016	14547.	22730.	1708	11.74%	21.5	4.6	174.4	1.2	266.5	1096.1	0.6	17
BIG SOIUX	1017	9172.	14331.	1121	12.22%	47.9	0.3	123.6	38.4	97.5	537.8	0.1	6.4
NORTH PLATTE	1018	20494.	32023.	358	1.75%	40.7	18.9	157.4	43.4	78.6	303.2	8.4	17
SOUTH PLATTE	1019	15214.	23772.	106	0.70%	22.4	6.9	113	0.7	56.6	92.4	4.4	10.8
LOWER PLATTE	1020	5291.	8268.	300	5.67%	95.9	0.9	40	0	67.5	127.7	9.5	32.7
LOUP RIVER	1021	9772.	15269.	447	4.57%	60	0	50.9	3.4	98	141	10.4	7.3
ELKHORN RIVER	1022	4403.	6881.	365	8.29%	26.8	0.3	2	0	4.8	127.6	0	6.6
LITTLE SIOUX	1023	6038.	9435.	1125	18.63%	56.3	0.3	32.6	0.1	7.8	99.1	1.1	5.8
NISHNABOTNA	1024	8646.	13510.	1039	12.02%	59.5	1.1	25.9	0.7	88.3	46.6	3	59.1
REPUBLICAN RIVER	1025	16065.	25102.	79	0.49%	42.7	0.5	8.4	8.4	65.7	22	1	4.2
SMOKEY HILL RIVER	1026	12725.	19884.	29	0.23%	30.6	3.6	39.2	3.8	34.4	10.2	0	0
KANSAS RIVER	1027	9729.	15202.	218	2.24%	58.1	1.5	35.8	0.2	136.3	37.6	2	9.2
CHARITON/GRAND R.	1028	7042.	11003.	1055	14.98%	30.4	7.6	20.5	9	51.2	66.4	10.1	71.7
GASCONADA/OSAGE R.	1029	11858.	18528.	564	4.76%	69	3	165.4	6.8	165.7	25.7	1.8	64.5
LOWER KANSAS RIVER	1030	6618.6	10341.	840	12.69%	54.3	1	35.8	7.7	105.1	8.9	1	71.5
TOTALS		301390	470923	40647	13.49%	1801	119	3721	290	3372	6892	1680	4030

(1) Hydric soils data derived from NRCS STATSGO data base.
(2) Wetlands inventory estimated using data from the NRCS 1992 NRI.

Federal Reservoirs

The Federal reservoirs throughout the basin played a significant role in reducing runoff and lowering stages in many ares hit hardest by the 1993 Flood. The existing Federal reservoirs located in the upland areas stored almost 30 million acre feet (ac-ft) of water during the flood event. It is estimated that existing farm ponds, erosion control structures and flood control reservoirs constructed with USDA assistance stored over 2 million ac-ft of water during the flood.

There are 76 flood control reservoirs in the upper Mississippi and Missouri river basins upstream of the mouth of the Ohio River. In addition to these flood control reservoirs, there are 25 locks and dams on the Mississippi River, and eight locks and dams on the Illinois Waterway that regulate the 9-foot navigation pools. The storage in the navigation pools is negligible during majorflood events. Most of the reservoirs, and the greatest potential to reduce flooding, are in the Missouri River basin. These structures range from the massive dams and reservoirs on the main stem Missouri River in Montana and the Dakotas to small headwater reservoirs on tributaries of both rivers. Some reduction to Mississippi River and Missouri River flood flows occurs for every flood due to reservoirs. During the last 20 years, flood stages have been reduced at St. Louis from two to seven feet, depending on the flood event. Without the reservoirs, most of the urban levees at St. Louis would have been overtopped during the 1993 flood. Reservoirs had significant impacts on the Missouri River, reducing the peak stages at Sioux City, S.D., at Omaha, Neb., at Kansas City, Mo., at Hermann, Mo. and at St. Louis. Reservoir effects on the Mississippi River upstream of the mouth of the Missouri were generally less than one foot upstream of Louisiana, Mo. and one to two feet downstream of Louisiana. The combined affects of reservoir storage in 1993 resulted in a 3.2 foot reduction in flood stage in St. Louis.

The severe flooding in 1993 resulted in damaging stages throughout the Mississippi and Missouri River Basins. These stages would have been higher in many locations if the system of flood control reservoirs had not been in place on various tributaries to the mainstem rivers. The available flood control storage onthe Mississippi river above St. Louis is about 5.8 million ac-ft. About 4.9 million ac-ft of this storage was used during the peak of the 1993 flood. The Missouri River has 67 reservoirs (Corps or Bureau of Reclamation projects) on its main stem and tributary stream watershed. There are six main stem (Corps) reservoirs controlling flow along the Missouri River. Although they were not within the July floodedarea, the six main stem reservoirs had a significant impact on reducing the peak stage experienced along the Missouri River downstream from Gavins Point Dam. The available storage on the Missouri River above St. Louis includes 13.4 million ac-ft of storage in tributary reservoirs and 13.1 million ac-ft of storage in the six Missouri mainstem reservoirs above Sioux City, Iowa. The tributary reservoirs stored approximately 10.5 million ac-ft during the peak of the flooding and the six mainstem reservoirs stored about 10.3 million ac-ft. In addition to the major Corps of Engineers and Bureau of Reclamation projects included in the above

TABLE SP-3
UPPER MISSISSIPPI RIVER BASIN
RESERVOIR STORAGE BY HYDROLOGIC UNIT

TOTAL FLOOD CONTROL STORAGE (1000 AC-FT)					
BASIN NAME	HYDRO UNIT HUC #	FEDERAL RESERVOIR STORAGE AC-FT(1000) (1)	TOTAL AVAIL PL 566 RESERVOIR AC-FT(1000)	TOTAL PL534/ASCS FARM PNDS AC-FT(1000)	MAXIMUM CHANGE IN RESERVOIR STORAGE (1993) (1)
MISS R. HEADWATERS	701	1491			341.7
MINNESOTA RIVER	702	145	4.6		89
ST. CROIX RIVER	703				
ZUMBO/ROOT R'S	704		8.1		4
CHIPPEWA R.	705		6.5		6.3
MAQUOKETA/PLUM R	706		10.9		5.4
WISCONSIN R.	707	400	14.8		327
IOWA/SKUNK R'S	708	475	1.0		487
ROCK R./FOX	709	409			325
DEMOINES R	710	2070	4.8		2068
WYACONDA/SALT	711	884	14.7		627
UPPER ILL.	712	13.6	5.3		15.1
LOWER ILL.	713		8.7		7
KASKASKIA/MERAMAC	714	1693	25.1		191
WHITE R	1014		5.3		4.2
NIOBRARA R	1015				
JAMES R	1016	329.2			117
BIG SOIUX	1017		14.6		11.6
NORTH PLATTE	1018		4.7		3.7
SOUTH PLATTE	1019		25.5		20.4
LOWER PLATTE	1020	139.5	33.3		53
LOUP R	1021				
ELKHORN R	1022		1.8		1.4
LITTLE SIOUX	1023		14.3		11.5
NISHNABOTNA	1024		78.4		62.7
REPUBLICAN R	1025	543.8	40.6		72.8
SMOKEY HILL R	1026	1312.7	51.9		918.2
KANSAS R	1027	5001.7	137.7		4638
CHARITON/GRAND R	1028	376.4	44.2		405
GASCONADA/OSAGE R	1029	5657.1	5.5		4207
LOWER KANSAS	1030	4859.4	15.6		4448
MISSOURI MAISTEM		13055			10310
SUB TOTALS		38799.4	578.0	1422	30303.9
TOTAL STORAGE		40799.4			
TOTAL 1993 STORAGE				31725.9	

(1) Data from COE 1993 Post Flood Reports.
Storage during 1993 flood includes total change in storage between the initial starting elevations and the maximum reservoir elevations experienced. Many of the reservoirs in the system stored and released water several times during the span of flood. These cumulative effects are not reflected in this table. It was assumed that 80% of the P.L. 566 storage was occupied for this summary.
NOTE: Almost all storage in Missouri Mainstem Reservoirs was in the Multi-Purpose storage areas. Only Oahe rose slightly into the annual flood control and multipurpose zone.

numbers, there are numerous USDA erosion control structures, State and private impoundments and farm ponds which account for an additional 2+ million ac-ft of storage throughout the basins. The volume of water temporarily stored or delayed in reservoirs above St. Louis during the period from June to August, 1993 is nearly 32 million ac-ft, or about 25 percent of the total volume of runoff measured at St. Louis during that same period. The storage affects of Federal, State and Private reservoirs during the 1993 flood are summarized in Table SP-3.

Agricultural Programs

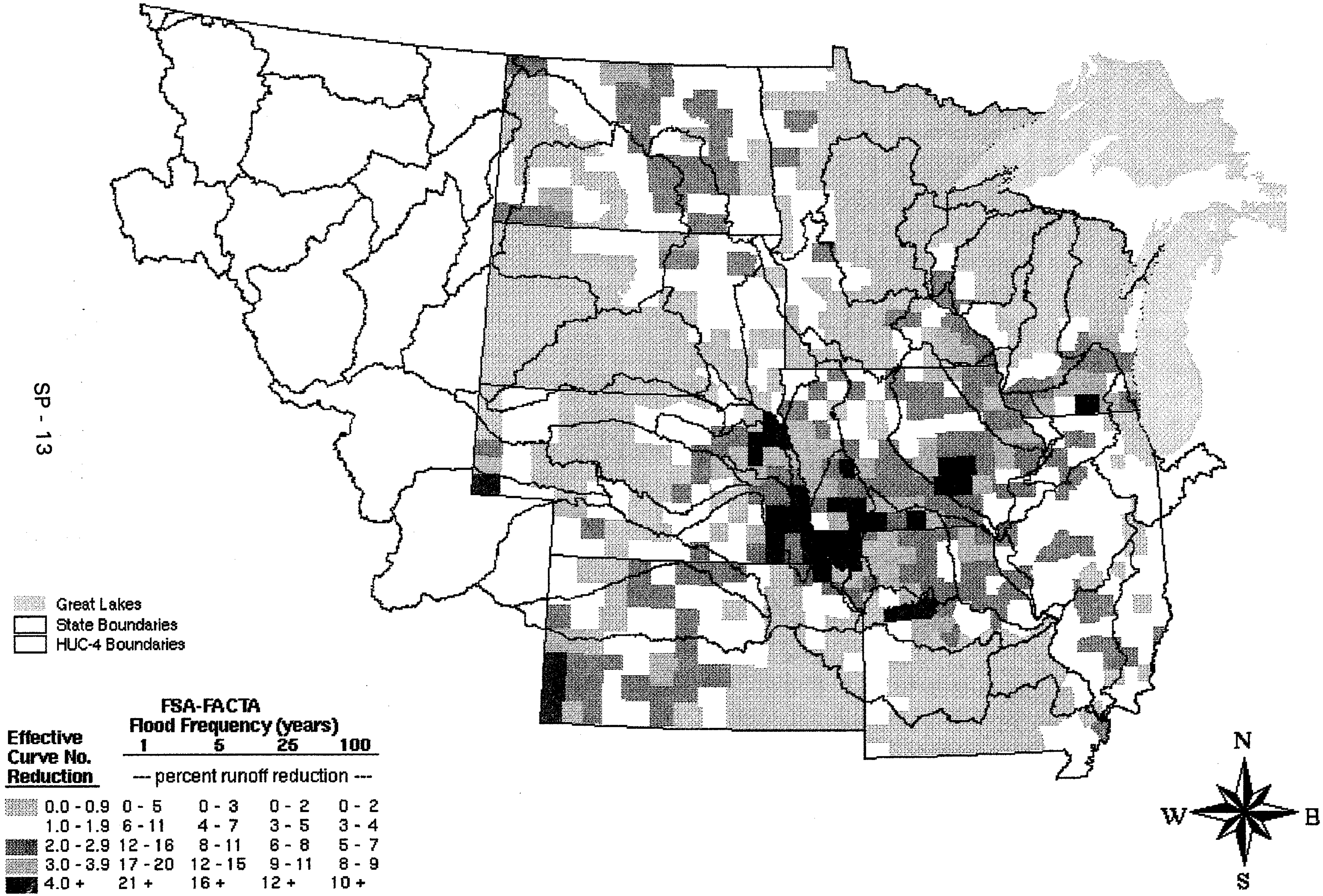
The impact of existing land treatment measures and natural storage was significantly larger than these structural measures due to the vast areas involved. The agricultural programs that have the largest potential for impacting flooding are the Food Security Act of 1985 (FSA) and the Food, Agriculture, Conservation, and Trade Act of 1990 (FACTA). These acts impose restrictions on persons who participate in certain USDA programs and who plant agricultural commodities on highly erodible lands or converted wetlands. The erosion provisions of the FSA and FACTA farm bills relate directly to surface water runoff. Practices such as residue management and reduced tillage increase infiltration of water into the soil and reduce the amount of surface water runoff. The reduction of surface water runoff will directly affect peak flows of streams, including flood events. Wetlands, especially potholes, provide surface storage of water and thus reduce peak flows during flood events.

Figure SP-1 shows the total reduction in runoff due to land management activities in compliance with FSA and FACTA. Figure SP-2 shows the reduction in runoff due to the conversion of cropland to grassland as part of the Conservation Reserve Program. These figures are modified versions of figures 7.1 and 7.2 of the SAST report and also show the principal hydrologic units (HUC) to better define which watersheds have the greatest concentrations of FSA/FACTA measures. The runoff reduction due to CRP is part of the total reduction due to FSA and FACTA. The Soil Conservation Service uses hydrologic runoff curve numbers (CN's) to indicate surface runoff potential from various soil-cover complexes. Change in land treatment affects curve number: that is, increases in soil surface cover resulting from FSA-FACTA reduce CN's; and, reduced CN's indicate higher infiltration and lower runoff. Reductions in runoff for four flooding frequencies were calculated over the 9-state area. For example, a curve number reduction in the range of 3.0 to 3.9 would result in a reduction in runoff of 12 to 15 percent for a 5-year flood (0.20 annual probability). Peak flood flow is related to flood runoff volume, but further modeling is needed to determine actual peak reductions due to timing of runoff within the basin and basin shape.

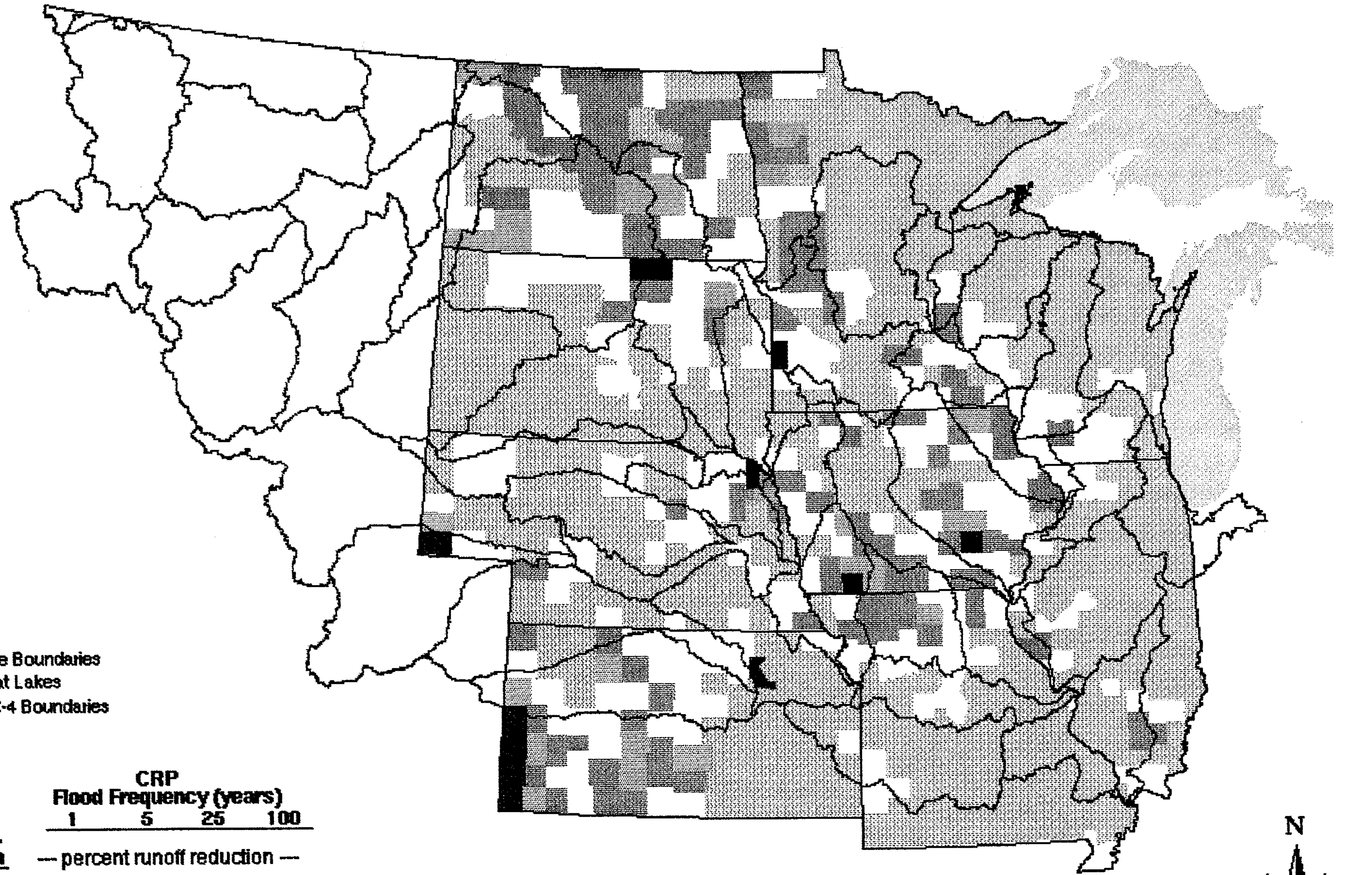
Additionally, the wetland conservation provisions of FSA and FACTA encourage the protection of wetlands, especially in the pothole region of the drainage basin. These pothole wetlands in the Prairie Pothole Region (Figure SP-3) provide surface storage of water and thus reduce peak flows during flood events. Programs, such as these entailed by FSA and FACTA, that encourage infiltration of water into the soil and thereby reduce runoff are important to fully attain the potential subsoil storage of water. These programs are especially important in the open drainage areas of the basin, since subsoil storage is one of the few (if not the only) nonstructural approaches to reducing stream flows. In formerly closed (depressional) drainage areas, programs such as the USDA Wetland Reserve Program and the DOI Small Wetland Acquisition and Partners for Wildlife Programs, which encourage reclamation of depressional or wetland areas that have been open ditch drained, are important to decreasing surface runoff and improving the water quality of the area.

UPPER MISSISSIPPI RIVER BASIN

FSA - FACTA RUNOFF REDUCTION



UPPER MISSISSIPPI RIVER BASIN
CRP RUNOFF REDUCTION



State Boundaries
Great Lakes
HUC-4 Boundaries

Effective Curve No. Reduction	CRP Flood Frequency (years) 1	5	25	100
	— percent runoff reduction —			
0.0 - 0.5	0 - 3	0 - 2	0 - 1	0 - 1
0.6 - 1.1	4 - 6	3 - 4	2 - 3	1 - 2
1.2 - 1.7	7 - 9	5 - 6	4 - 5	3 - 4
1.8 - 2.3	10 - 12	7 - 8	6 - 7	5 - 6
2.4 +	13+	9 +	7 +	6 +

Wetlands of the Upper Mississippi River Basin

Wetlands in the upland areas of the basin are discussed with respect to changes in wetlands from presettlement to the present, to their value for biodiversity and for water quality, and to their potential use for flood redirection in the basin. Presettlement vegetation consisted of deciduous hardwood forests in the eastern Ozark Plateau parts of the basin, tall-grass prairies in the central part, and mixed and short-grass prairies in the western part. In the prairie region, woodlands occurred in riparian zones and around upland wetlands. The amount of presettlement wetlands in the basin is estimated at 58 million acres (Kusler, 1993; Table 4.2). Presently there are about 23 million acres of wetlands remaining in the basin (Kusler, 1993). The loss of 35 million acres of wetlands has mostly been a result of agricultural drainage (Kantrud, 1989; van der Valk, 1989), and channel modification and flood control (Funk and Robinson; 1974, Eckblad, 1988; Jahn and Anderson, 1986). Table SP-4 displays an estimate of current wetlands status in the study area by state.

TableSP- 4
Percentage of wetlands in States circa 1780 and present (after Dahl 1990)

STATE	PERCENTAGE OF WETLANDS 1780	PERCENTAGE OF WETLANDS PRESENT
ILLINOIS	22.8	3.5
IOWA	11.1	1.2
MINNESOTA	28.0	16.2
MISSOURI	10.9	1.4
NORTH DAKOTA	10.9	5.5
SOUTH DAKOTA	5.5	3.6
WISCONSIN	27.3	14.8

Hydric soils classifications are also another means of estimating the original extent of the great northern prairie wetland area before the influence of man so radically changed it. Figure SP-4 shows the extent of the wetlands based on hydric soils classifications from the NRCS STATSGO data base. Table SP-2 also tabulates the total estimated hydric soils acreages by 4 digit hydrologic unit (HUC). The map is representative of the original wetland concentrations in the study area. Notice the correlation between the prairie pothole region in Figure SP-3 and the higher concentrations of hydric soils show in the hydric soils map. The depressional areas in the prairie pothole region are quite extensive and these soils exhibit hydric properties. In many areas of the pothole region such as the DesMoines and Minnesota River basins, over 90 percent of the hydric soils are drained or used for agricultural purposes.

Upland Agricultural Drainage

Over 60 percent of the depressions in the closed drainage areas of Iowa, Minnesota, and Illinois are tile or open ditch drained (USDA-SCS NRI data, 1982). Agricultural drainage systems are installed to lower the water table or remove surface water to (1) provide trafficable field operations, (2) protect crops from excess water conditions, and (3) control salinity in irrigated arid and semi-arid areas (Skaggs and others, 1994). Skaggs and others (1994) describe two stages of land modification for agricultural use and resulting changes in surface runoff and peak flows.

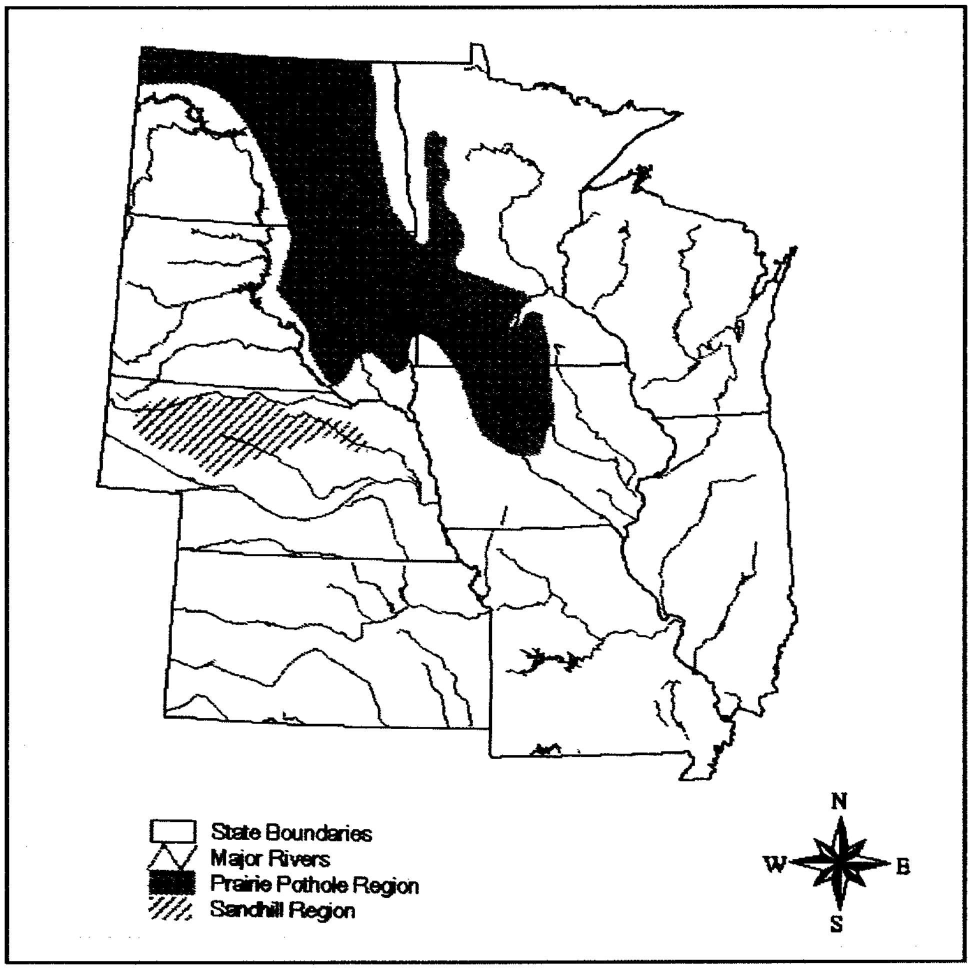
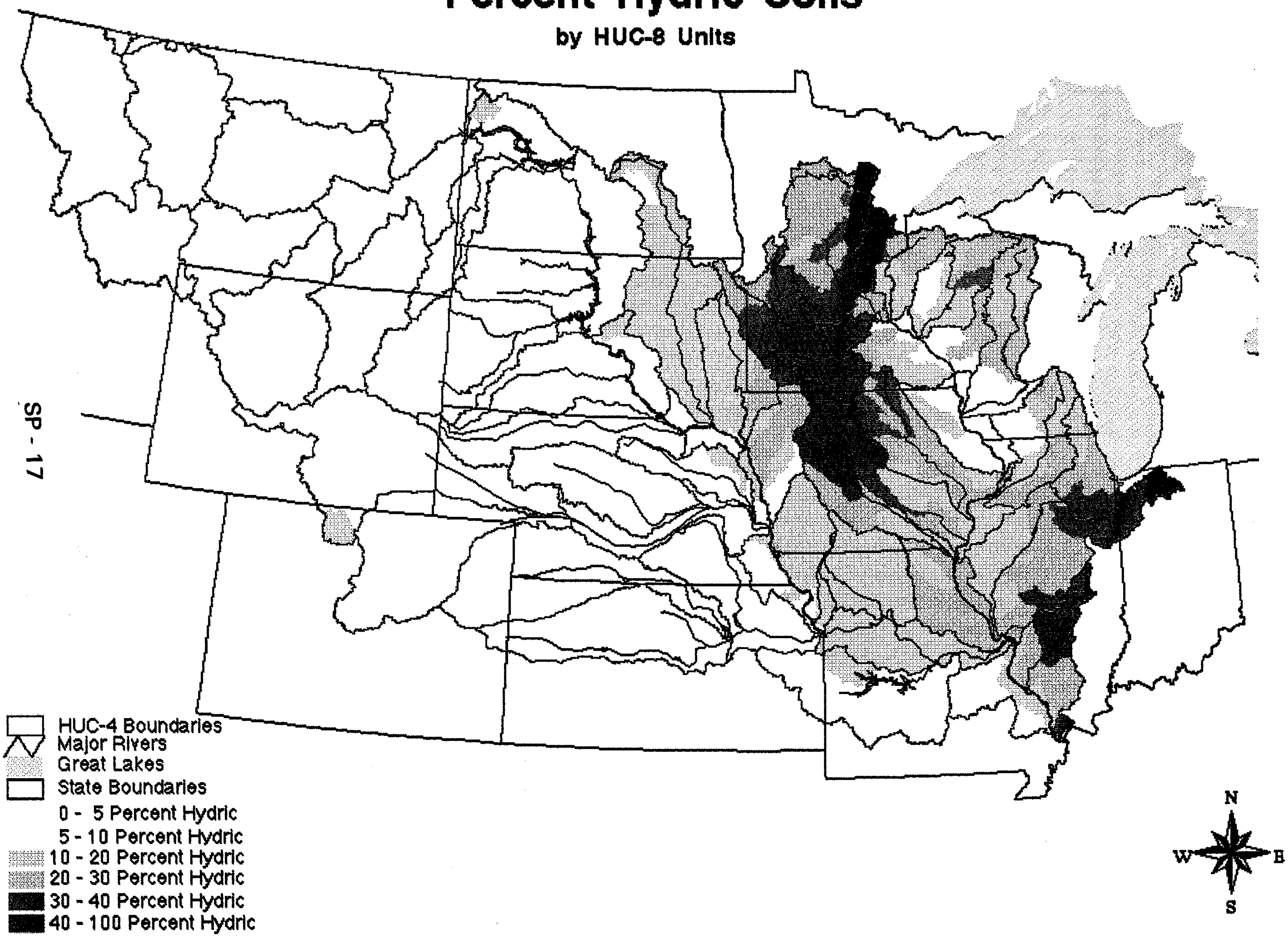


Figure SP-3. Northern Prairie Wetlands.

Percent Hydric Soils

by HUC-8 Units



During the first stage, the land is cleared or plowed. This process decreases infiltration, increases the wetness of the land by lowering evapotranspiration, and increases surface runoff. Open ditch drains are then constructed to act as outlets for the removal of excess water. Since they are direct conduits to streams, these open ditches eliminate a high percentage of the surface storage of the depressional areas and can result in increased peak flows. Research results have shown that drainage improvement (surface drainage) plus land conversion from natural vegetation to row crops increases peak runoff rates at the field edge by a factor of 200 to 400 percent over the rates in the natural condition (Skaggs and others, 1994). Improved drainage also reduces time to runoff peaks.

During the second stage, tile drainage systems are developed for each field of a farm. The outlets for these systems are in the open ditches constructed in the first stage. Once installed, subsurface tile systems lower the water table and remove water from depressional areas at a commonly used design rate of 1.25 cm/day (1/2 inch/day). Traditional subsurface drainage systems lower the peak flows created in the first stage of drainage activity by 35 to 55 percent (Skaggs and others, 1994). By increasing storage capacity in the soils and storage in drained depressions, traditional tile results in controlled removal of water from depressions over a period of days and decreases the peak flows in streams. However, the combined effect of agricultural drainage increases flow in streams over surface flow from natural systems. Exact figures depend on topography, field drainage systems, and distance to open ditch systems.

Some tile drainage systems have standpipes from the soil surface to the tile line; they are used to lower the risk of crop failure from ponding of water in depressional areas. These standpipe tile drains are more efficient in removing water than traditional tile since the standpipe provides an open conduit from the surface ponded water to the tile line. Many tile drains to depressions have standpipes in the Des Moines Lobe area of Iowa and Minnesota (G. Schellentrager, personal commun., 1994). Maidment (Proceedings of SAST Hydrologic Modeling Workshop) found that, in the Walnut Creek watershed in Iowa, standpipe drains act more like open drainage systems, transmitting water directly to stream channels at rates that are near those of surface drainage. However, SCS sources (R. Bartels and D. Miller, written commun., 1994) note that the underground tile joined to the standpipe has limited capacity and that these drains have a throttling effect on flow from depressional areas. Further study is needed to determine the impact of standpipe tile drains on stream flow.

Traditional tile drainage systems help keep the depressions empty and thus provide the maximum amount of surface water storage, so that full storage capacity is realized during rainfall events. The tile systems then drain the depressions in a controlled manner over a period of a few days. This drainage has the effect of broadening the peak of the hydrograph and decreasing flow rates in comparison with open ditch systems. Tile systems with standpipes, however, drain water to streams at rates that may approach those of surface drains, depending on the size of the tile and the magnitude of the storm, and therefore may not be effective for surface storage of water.

Flood Reduction Alternatives

The definition of runoff relationships through rainfall amount, rainfall intensity, timing of a series of storms, topography, land use, antecedent conditions, drainage network and consideration of existing upland and valley storage is very complex. It can be the subject of considerable difference of opinion. Unless evaluations are done using detailed, systematic process, with several calibrations during the process, the results can not be defended based on scientific procedures. The use of different hydrology runoff models to evaluate various combinations of these runoff relationships occurs throughout the basin, but none evaluate all of the processes over the entire watershed impacted by the 1993 Flood.

It was determined very early in the assessment process that time and funds were not available to perform comprehensive deterministic hydrologic studies on the entire area impacted by flooding in 1993. The approach used in this assessment was to use the available information developed for the SAST report and any additional information readily available from other sources such as COE, NRCS, USGS and the FWS. This information would help define physical relationships between runoff volume and the structural and non-structural measures typically used to reduce runoff volume.

Upland Flood Control Measures

Control of runoff in the upland watershed is accomplished through both structural and non-structural measures. These measures include land treatments that affect the soils infiltration rate, the soil moisture retention capacity and protection or restoration of natural flood water storage areas. Wetlands, or construction practices like terraces, farm ponds, erosion control structures or flood control reservoirs all have capacity to store excess runoff. The impacts of existing land use and upland treatments on the 1993 flood were estimated using information included in the SAST report along with data from the NRCS, COE, NOAA, NWS and the USGS.

Several alternatives were considered to test the sensitivity of different types of upstream flood control measures. The "Existing condition" alternative identifies the base condition which treats all land use and upland storage in Federal Reservoirs as it existed in 1993. The "Without Federal Reservoirs" alternative is used to identify the effects the 1993 storage in these reservoirs had on reduced flood stages in the downstream floodplain. The "With Revised Reservoir Operations" alternative reviews existing operating plans to determine if adjustments to these plans would have further reduced flooding in the floodplain. The "Runoff Reductions of 5 and 10 Percent" alternative examined assumed volume reductions to the tributary hydrographs through additional upland land use changes or storage measures. These reductions were based on very general information at this time, and additional detailed evaluations will be needed to determine optimum combinations of non-structural and structural measures required.

Existing Conditions

The existing conditions scenario evaluates all land use conditions and reservoir operations as they actually existed in the 1993 flood. Conditions conducive to producing extreme runoff existed withantecedent soil moisture in the 70-90 percent of soil capacity throughout most of the basin and precipitation patterns were well above normal over most of the basin. These antecedent conditions reduced the ability of the upland features such as wetlands, flood storage reservoirs and depressional areas to store runoff. The extreme soil moisture contents significantly reduced the soils ability to store additional water. Also, the extremely wet, cool conditions had drastically inhibited spring planting in many areas, resulting in little vegetation in thefields of this heavily farmed region. This further exacerbated the flood conditions as it reduced the soil moisture holdingcapacity and left little vegetation for evapotranspiration. The soil recovery rates (the ability to percolate water to the ground water table and remove topsoil moisture through evapotranspiration) were also reduced due to the persistent wet cycle and lack of vegetation.

The rainfall events that contributed to the severe flooding in the study area were intense storms which occurred over short periods of time on soils near saturation. These storm durations range from several hours to 1-2 days with rainfall intensities exceeding 1 inch/hour during portions of these storms. The soils over most of the basin had reduced infiltration rates thatwere well below 0.1-0.15 inches/hour that could normally be expected during this time of year. Therefore, rainfall exceeding these reduced infiltration capabilities went into direct runoff. The natural ability of the uplands to attenuate the runoff through depressional and wetland storage has been depleted through the years by agricultural and urban drainage. Also, the spring antecedent conditions and the succession of storms which pelted the midwest would have further reduced storage in these natural buffers. The combination of these extreme conditions led to excessive runoff throughout the basin.

The SAST report included information on the impacts of the many upland treatment programs administered by the USDA and how these programs may have contributed to reduced flood volumes in 1993. These "*non-structural*" programs include the Conservation Reserve Program (CRP), Wetlands Reserve Program (WRP) and erosion control programs such as terracing and residue management. The Natural Resources Conservation Service (NRCS) estimates that the combined effects of these programs within the state of Iowa amounted to reducing runoff volume by about 700,000 ac-ft. These uplands reduction measures affect each rainfall event that occurs to some extent as both storage of runoff and soil infiltration are impacted.

Nonstructural Flood Reduction

Nonstructural methods of flood reduction have long been encouraged in the United States. Often the methods have been associated with other goals such as capturing the maximum amountof rainfall for agricultural crops, or restoring prairie pothole wetlands for the use of waterfowl, or preventingerosion. These methods, while practiced to accomplish other goals, can provide flood reduction benefits. For example, the NRCS for many years has encouraged treatment of farmland to reduce runoff and erosion, both of which affect flooding in a basin.

The water purification function of wetlands is now better understood and is being investigated and

encouraged as a method of treating agricultural wastewater prior to returning it to streams. Some communities and organizations are also moving to restore and create wetlands as a part of a basin-wide policy to improve water quality. One such activity is the Redwood River Basin Association's effort to restore up to 5,000 acres of wetlands for water quality improvement (Cooper, 1994). These wetlands should not only improve water quality, but can also have an impact on flooding if properly restored, as will be discussed in a following section of this appendix.

With proper education and techniques, methods currently promoted for other purposes can be used or adapted to assist in flood reduction. For example, an awareness of the relationship of wetland types and their effect on flooding can result in actions in which wetlands restored for wildlife can assist in local flood reduction as well. Thus, wetland restoration not only can improve water quality from agricultural watersheds, but also can provide habitat for wildlife and, to some extent, reduce downstream flooding as well.

The construction of wetlands on the floodplains or in the watershed, however, can only reduce peak flooding to the maximum amount of their storage capacity. In large storms, the water volume stored in wetlands that form a small portion of the watershed can be negligible, but in watersheds with large amounts of wetlands in the upland, wetlands may have an appreciable effect on flooding. The effect of wetlands depends on the volume of wetland storage relative to the volume of the flood, the location of the wetland, and the duration of the flood.

SAST Model Watersheds

The SAST was initially charged with evaluating both structural and nonstructural approaches to river basin management. It was immediately apparent that the best place to apply nonstructural methods for flood reduction would be in the uplands because of the comparatively greater amount of lands available in the upland than in the floodplain areas. In an effort to evaluate nonstructural flood reduction measures in the uplands, a preliminary evaluation of upland land treatments, wetlands, and detention storage was conducted to view the range of reductions in flood peaks that might be possible on various types of watersheds. In order to apply the results as widely as possible, watersheds were selected to represent basins that differed in terrain as widely as possible.

Since time was the critical constraint, the selection of watersheds was driven by the existence of calibrated models for watersheds that were to be evaluated. The four selected watersheds represent distinct types of landscapes - a steep basin, a low relief pothole basin, a low relief basin with well defined drainage, and a relatively high relief basin that has been drained for agriculture. It should be noted that while the Whitebreast Creek and Redwood River studies were rather detailed (especially the Whitebreast Creek model), the Boone River and West Fork Cedar River models used much less detail due to time constraints and the lack of pre-existing detailed models.

The SAST recognized that since different models were used for the study basins, the results might not compare exactly. The general trends, however, would be identifiable for each watershed and the relative differences could be noted between the watersheds. Time constraints and the need to answer important questions regarding wetlands and flood reductions, SCS land practices, and detention basin effects, dictated the use of two different hydrologic models. Additionally, four groups of modelers were used in the studies to facilitate timely completion of the modeling. The basins and associated modeling groups are: Boone River - Corps of Engineers, Hydrologic Engineering Center (HEC, 1994); West Fork Cedar River - Corps of Engineers, Waterways Experiment Station (WES, 1994); Whitebreast Creek - USDA-SCS, Iowa (SCS, 1994a); and Redwood River - USDA-SCS, Minnesota (SCS, 1994b).

The SCS curve number method was used for all studies where land management practices were evaluated. Storm durations of 24 hours were used, with the exception of the Boone River and Redwood River basins where a 24-hour storm would not account for the long travel time of about 90 hours to reach the basin outlets. The storm duration used for the Boone River was a 4-day (96 hour) storm and for the Redwood River basin the duration was a 6-day (144 hour) storm. All model runs used antecedent moisture condition II for the start of modeling conditions. Condition II is defined as the average soil moisture condition prior to the annual flood event. For the 1993 flood, antecedent condition III existed in most areas. Condition III indicates near saturated soils prior to the storm and gives significantly higher runoff than antecedent moisture conditions I (dry enough to cultivate) or II. Time constraints precluded modeling antecedent conditions I and III. Additionally, standard modeling practice uses antecedent condition II as a starting condition for the modeling of standard storm events.

The objective of the modeling was to show the effect of various management, land use, and storage practices on the outflow hydrographs for differing types of basins. Alternatives selected for Boone River, West Fork Cedar River, and Whitebreast Creek watersheds were:

1. maximizing wetland storage in upland and/or floodplain areas as applicable,
2. converting cropland to CRP as available,
3. maximizing infiltration by using all applicable land treatments such as conservation tillage, terraces, and permanent cover, for highly erodible lands (HEL)
4. installing flood prevention structures as applicable - i.e., traditional SCS small (P.L. 566) watershed structures to temporarily store water and release it slowly,
5. a combination of all nonstructural practices - i.e., alternatives 1, 2, and 3, and
6. a combination of all possible alternatives - alternative 5 plus alternative 4 to demonstrate the maximum possible reduction for the watershed without the inundation of large areas for medium to large reservoirs.

Since the objective of the studies was to determine maximum effects, these treatments and options were assumed to be applied to 100 percent of the basin --- an unrealistically high assumption. Although the actual percentage of coverage for the basin would be lower, this method gives the maximum effect for each alternative.

Not all land treatments are readily adaptable to all watersheds. The construction of detention basins, for example, is most economically feasible in watersheds with incised drainageways. A tour of the West Fork Cedar River basin revealed very few possible sites for detention basins, and since the available sites were deemed too few to provide a significant impact, this option was not modeled for the basin. Similarly, too few wetland areas were found in the West Fork Cedar River basin to have an impact. It should be noted that an off-stream wildlife site exists in this basin, but a shallow depth (approximately 4-5 ft) would cover several thousand acres. This makes the construction of detention basins or wetlands in the basin a very land-intensive undertaking. Detention structures were not evaluated on the Boone River since few, if any, sites exist in the watershed, while some potential for wetland development or restoration would exist.

Results of SAST Watershed Studies

The results of the four watershed studies are presented in Table SP-5. Three of the watersheds show a trend of reducing effect with increasing storm return period for the combination of all modeled alternatives, while the Whitebreast Creek model shows the opposite effect for 5-year or greater storms. It should be noted that when the design capacity of detention basins is exceeded, the effectiveness of the basins for flood peak reduction decreases to zero as the amount of overflow increases. Analysis of Table SP-5 shows that the maximum infiltration option (FSA) at Whitebreast Creek results in a rather variable reduction that is nearly as high for the 100-year storm as it is for the 1-year storm. This result and the results for the detention basins are probably due to the timing of runoff in the watershed and the interaction of the detention storage and storage in terraces. The results have been closely scrutinized by the Iowa SCS State Office staff and are believed to be accurate. This variation is an excellent example of the reason why results from one basin cannot be directly applied to other basins.

Without Federal Reservoirs

The analysis of the 1993 flood event without the federal reservoirs storage was accomplished by determining the discharge hydrograph at each site without the storage effects of the reservoir. These unregulated hydrographs were then routed downstream to determine the effects on peak discharges and stages at critical locations. The hydrographs were routed to the Missouri and Mississippi mainstems from the upstream tributaries using hydrologic routing and the UNET model was used to route the hydrographs through the floodplains to determine resultant water surface profiles. The flood storage in federal reservoirs had significant impacts on flood stages during the 1993 flood on the Mississippi River from Grafton to Cape Girardeau and on the Missouri River from Gavins Point Dam to the mouth at St. Louis. Flood stages in these reaches would have been several feet higher if the federal reservoir system had not existed.

Table SP-5
SAST Watershed Analysis Results

Percent (%) Flood Peak and Volume Reduction by Watershed and Treatment (Revised)								
Return Period	Boone River		West Fork Cedar River		Whitebreast Creek		Redwood River	
	Peak	Volume	Peak	Volume	Peak	Volume	Peak	Volume
Floodplain Wetlands								
1	5	0			1	***	6	1
5	3	0			1		5	1
25	2	0			2		3	1
100	2	0			0		3	0
Upland Wetlands or Potholes								
1	9	7					23	2
5	8	4					15	3
25	7	1					11	4
100	5	0					10	2
Conservation Reserve Program (CRP)								
1	3	2	7	6	4			
5	1	1	5	4	4			
25	1	1	4	3	4			
100	1	1	3	3	4			
Maximum Infiltration (FSA) - Includes CRP Reductions								
1	6	4	15*	14	21			
5	3	3	11*	10	15			
25	2	2	8*	8	18			
100	2	2	7*	7	20			
Detention Structures								
1					8		26	4
5					15		16	4
25					27		12	5
100					28		11	3
Total of All Applicable Treatments								
1	18	12	15*	14	29		27	11
5	14	8	11*	10	21		21	12
25	12	4	8*	8	37		17	12
100	9	2	7*	7	40		16	11

* In the original SAST Table these numbers were incorrectly reported without the CRP effects which are included in the FSA programs and are included in other watersheds.

** This table is taken from the Preliminary Report of the Scientific Assessment and Strategy Team with the Total of All Treatments for the West Fork Cedar River Revised to the correct peak values.

*** Adequate data could not be obtained to determine volume reductions for Whitebreast Creek.

Analysis by Omaha District was performed to assess the affect of reservoir storage on peak flow rates during the 1993 event. Within the Omaha District, major Federal reservoirs include the six mainstem dams on the Missouri River upstream of Gavins Point Dam at river mile 811.1. The Missouri River Division Reservoir Control Center (RCC) annually computes the without reservoir hydrograph at Gavins Point Dam based on routed upstream inflows. UNET modeling was performed employing the without reservoir flow hydrograph computed by RCC for inflow into the model instead of the actual 1993 reservoir releases. All other parameters were unchanged from the base condition. The without reservoir hydrograph computed by RCC at Gavins Point Dam did not contain any large peaks flows during the 1993 event. Discharge generally varied from 60,000 - 90,000 cfs for a 3 month period. Essentially, the without reservoir hydrograph is equivalent to adding substantial base flow to the Missouri River for the 1993 event. Discharges for the Kansas River at Desoto without reservoir holdouts would have been approximately 266000 cfs as opposed to 170000 cfs for the regulated condition. Downstream at Herman, the unregulated discharge would have approached 850000 cfs as opposed to 750000 cfs for the regulated condition.

Elimination of all Federal flood control storage in the entire Upper Mississippi River system would result in an increase in stage at St. Louis of about 3.2 feet. Tables SP-6 and SP-7 show the increased stages that would have occurred without Federal reservoir storage at various stations along the Mississippi and Missouri Rivers. The stage increases are less than 0.5 feet above L/D 22 on the Mississippi, but increase to 3.3 feet when the affects of reservoir storage on the Salt and Illinois Rivers are added todischarges. Stage increases on the Missouri with reservoir holdouts added in vary from near zero to over (5) feet in the reach from Omaha to St. Charles depending on effects of the agricultural levees. This reach is discussed in greater detail in the Omaha Section of this Appendix.

TableSP- 6
Kansas River - 1993 Flood
Actual and Unregulated Stages & Discharges

Stream Gage			Actual			Unregulated		Stage Reduction by Federal Reservoirs (ft.)
Place	River Mile	Flood Stage (ft.)	Date	Discharge (cfs)	Stage (ft.)	Discharge (cfs)	Stage (ft.)	
Fort Riley	168.9	21.0	7/26/93	87,600	27.9	200,000	35.0	7.1
Wamego	126.9	19.0	7/26/93	199,000	27.3	258,000	28.9	1.6
Topeka	83.1	26.0	7/25/93	170,000	34.9	260,000	37.1	2.2
Lecompton	63.8	17.0	7/27/93	190,000	24.7	282,000	26.9	2.3
Desoto	31.0	24.0	7/27/93	170,000	26.9	266,000	31.4	4.5

Data from 1993 Post Flood Report, Appendix E.

Table SP-7
Missouri River - 1993 Flood
Actual and Unregulated Stages & Discharges

Stream Gage			Actual			Unregulated		Stage Reduction by Federal Reservoirs (ft.)
Place	River Mile	Flood Stage (ft.)	Date	Discharge (cfs)	Stage (ft.)	Discharge (cfs)	Stage (ft.)	
St. Joseph	448.2	17.0	7/26/93	335,000	32.6	461,000	33.0	0.4
Kansas City	366.1	32.0	7/27/93	541,000	48.9	713,000	54.0	5.1
Waverly	293.4	20.0	7/27/93	633,000	31.2	700,000	32.4	1.2
Boonville	197.1	21.0	7/29/93	755,000	37.1	820,000	38.5	1.4
Hermann	97.9	21.0	7/31/93	750,000	36.2	852,000	39.8	3.6

Data from 1993 Post Flood Report, Appendix E. Revised stages based on FPMA UNET modeling.

Revised Reservoir Operation

Reservoir operating plans are designed to optimize benefits for the projects intended and authorized purpose. These operating plans are usually keyed to specific benefits in many cases which are local in nature. These optimizations are based on either historic or synthetic flood simulations or some downstream contol point to determine the optimum operating conditions which maximize both upstream and downstream benefits. Also, releases are made based on known hydrologic conditions or forecasts. In the case of the 1993 flood, the downstream hydrologic conditions were a moving target for reservoir operators. Critical rainfall events downstream of these projects occurred after upstream forecasts were made. In some cases, upstream reservoir releases were made several days before significant rainfall events downstream further exacerbated conditions at key downstream damage locations. In most cases, revised operating plans would only benefit had there been foresight of the conditions that changed downstream after releases were made days or in some cases weeks earlier. Table SP-3 displays the storage characteristics of some of the basins with the most significant reservoir influences and indicates the amount and percent of flood storage utilized. The table indicates that storage effects varied throughout the basin and not all flood control storage was required to meet downstream flood control commitments.

During the July 1993 peak flooding period, reservoir releases from Gavins Point Dam averaged 8000 cfs. Release volume from Gavins Point Dam totaled 2.06 million acre-feet from June through August, 1993. Reservoir release rates corresponded with minimal releases required for downstream water uses. The minimal flow released from Gavins Point Dam had no effect on downstream flood levels. Further reduction of reservoir releases during the 1993 flood event would not have been practical or beneficial. Refer to the 1993 - 1994

Missouri River Mainstem Reservoir Annual Operating Plan report for details regarding system inflow, pool levels, and operation of the mainstem reservoirs.

The analysis of reservoir operation plans is complex, with multiple uses and interests competing for water benefits. Revising operating plans to better optimize the flood storage benefits for the 1993 flood could provide slight reductions in local flooding below the reservoirs, but only for the 1993 hydrograph shape and volume. Detailed analysis of revised operation of some reservoirs for the 1993 flood could reveal that there are some operating plans that could be improved based on lessons learned. However, this process is lengthy and beyond the scope of this assessment.

5 and 10 Percent Reductions

The volume of runoff is the most critical and controlling factor for defining flooding in the floodplains of the Mississippi and Missouri Rivers as the contributing area to the floodprone area becomes large. The runoff hydrograph shape is sensitive to upland retention measures in the upland floodprone areas. However, as these upland hydrographs are combined with other tributaries and travel downstream; the shape of the hydrograph is not as sensitive to individual uplands retention measures. Since this assessment concentrates on the floodprone areas of the larger downstream floodplains, the evaluation of the impacts of various upland retention measures on local hydrograph distribution was determined not to be critical. Therefore, impacts of runoff reduction measures addressed in this assessment will assume that uniform volume reducions of tributary hydrographs can be applied without significantly affecting the credibility of the floodplain sensitivity analysis.

The 5 and 10 percent volume reductions are used to test the sensitivity of the floodplain water surface profiles to changes in tributary hydrograph volumes for the 1993 flood. The reductions are intended to represent changes in upland watershed land use through either structural or non-structural measures. Since the measures were to be weighed against 1993 flood conditions, the volume reductions and measures assumed had to account for the extreme antecedent conditions that existed in these watersheds during the critical months of June through July. The tremendous volumes of runoff experienced throughout the basin when multiplied by 5 or 10 percent reduction factors result in very large volumes in some watersheds. Because of the tremendous volume involved in the 1993 flood event, it was determined at meetings attended by other state and federal agencies that the volume reductions on tributary hydrographs should not exceed 10 percent of the recorded 1993 runoff. The volume reductions in these watersheds would require a combination of both structural and non-structural measures to achieve up to 10 percent volume reductions for the 1993 flood.

The non-structural measures assumed to achieve the 10 percent volume reductions included wetland restoration and FSA/FACTA measures. Table SP-8 displays several tributaries and their relationships between the 1993 runoff volume and the soils classified as either hydric or wetlands. The total wetland acreages displayed are based on soils classifications and are more than double the USFWS National Wetlands Inventory estimates.

However, the wetland acreages listed in the table may be reasonable estimates of the wetlands available for restoration.

Assuming wetland restoration and management could have achieved an additional 1 foot of storage under the 1993 antecedent conditions, almost 1 million acres (more than 10% of the total watershed) of wetland restoration on the Des Moines River Basinwould be required to accomplish all of the 10 percent volume reduction. On the Kansas River, wetland restoration could accomplish only a small percentage of the required volume reduction. The flooding in the James River basin was less severe in 1993 and wetland restoration could possibly achieve a large percentage of the volume reduction. The data in the table indicates that in most cases, less than 5 percent of the volume reductions are achievable under 1993 antecedent conditions.

Table SP-8
Examples of 1993 Flood Volume -VS-
Available Wetland Restoration Acres

BASIN NAME	HUC #	TOTAL AREA ACRES (1000s)	HYDRIC[1] SOILS ACRES (1000s)	WETLANDS[2] ACRES (1000s)	1993 RUNOFF VOLUME (1000 AC-FT)	10% OF 1993 RUNOFF (1000 AC-FT)
MINNESOTA R.	702	10840.5	4318	1314	6369	636.9
DEMOINES R	710	9195.2	3339	425.6	9990	999.0
ILLINOIS R.	712/713	18450.7	4991	575.3	9916	991.6
JAMES R	1016	14547.3	1708	1380.2	915	91.5
PLATTE R.	1018-1020	40999.0	764	808.8	4291	429.1
KANSAS R	1025-1027	38521.0	326	322.6	10042	1004.2

1. Hydric soils estimated from NRCS STATSGO data. The hydric soils are an indication of the original pre-development wetland acreages.
2. Wetlands estimated from 1992 NRCS NRI data. These estimates are based on soil characteristics and not on NWI (National Wetlands Inventory, USFWS) characterizations. In most cases the total undisturbed wetlands acres are less than half of the NRI estimated acreages.

Other non-structural measures or land treatments considered include maximizing infiltration through use of conservation practices and cropland conversion. Structural measures would include the traditional SCS small (P.L. 566) watershed structures and larger flood storage structures where necessary. The time available to conduct the assessment did not permit a detailed analysis of land use for each sub-watershed. This would require comprehensive deterministic hydrologic models to measure all the physical processes related to these changes. Models exist which represent small portions of the basin, but not tothe extent that they would provide appropriate coverage to perform detailed, comprehensive analysis on this very diverse landscape. However, data provided in the SAST report and existing COE, NRCS and other federal agency data provides a level of understanding of these physical features and processes such that estimates on how land use changes will affect volume relationships can be developed to the level of detail commensurate with this assessment's objective.

These case studies evaluated the effects of combinations of land use changes on four selected watersheds

which represent four distinct landforms in the Upper Mississippi River Basin. The four landforms included a steep basin, a low relief pothole basin, a low relief basin with well defined drainage and a relatively high relief basin that has been drained for agriculture. The studies were not conducted using the same hydrologic model, but general trends were identifiable and relative differences could be noted from the studies. These studies indicated that reductions in flood peaks from upland land treatments can be influenced by many factors. The floodplain geomorphology, hydrologic characteristics, antecedent conditions and precipitation distributions are some of the factors. The studies also indicate a trend toward decreasing influence on flood peaks as precipitation or flood recurrence interval increases. Where land use changes may reduce flood peaks by between 25-50% for a flood with a return period of 2-5 years, the same changes may only reduce peaks by 10% or less for floods with return periods of 100 years or greater.

Wetland restoration has proven to be an effective flood reduction measure in the upper watershed areas where the localized affects are most pronounced. The SAST case studies indicate that flood peaks can be reduced significantly for fairly frequent flood events. However, wetland restoration measures would have had drastically reduced affects on flood volumes under the antecedent conditions and the extreme floods conditions that existed throughout most of the watershed in 1993. It is questionable whether restoration of drained depressional areas would contribute to flood reduction under these extreme conditions. It can be argued that these drained depressions actually provide greater flood reduction benefits by preserving the depressional storage for the most extreme rainfall events through drainage of antecedent events. The drainage of wetlands is a very complex hydrologic issue with broad social, political, economic and environmental impacts. This assessment addresses the restoration of wetlands as one of a combination of the uplands measures used to achieve the 5 and 10 percent volume adjustments in the upper watersheds. Additional studies and inventories are required to better define the current wetlands status for the entire Upper Mississippi River Basin and will indicate where in the basin wetland restoration would have had the most influence on the 1993 flood.

In summary, the volume reductions assumed for the floodplain sensitivity analysis are 5 and 10 percent of the 1993 runoff volume from all tributaries of the Mississippi River above Cairo, Illinois and below Sioux City, Iowa on the Missouri River. The adjustments are not based on specific flood reduction measures or combinations of measures for each tributary. Instead, it is assumed that there is a combination of both non-structural and structural changes that could achieve these reductions. It is also assumed that the 10 percent volume reduction is an upper bound on what is reasonably achievable under the extreme antecedent conditions and flood conditions that existed throughout most of the watershed in 1993.

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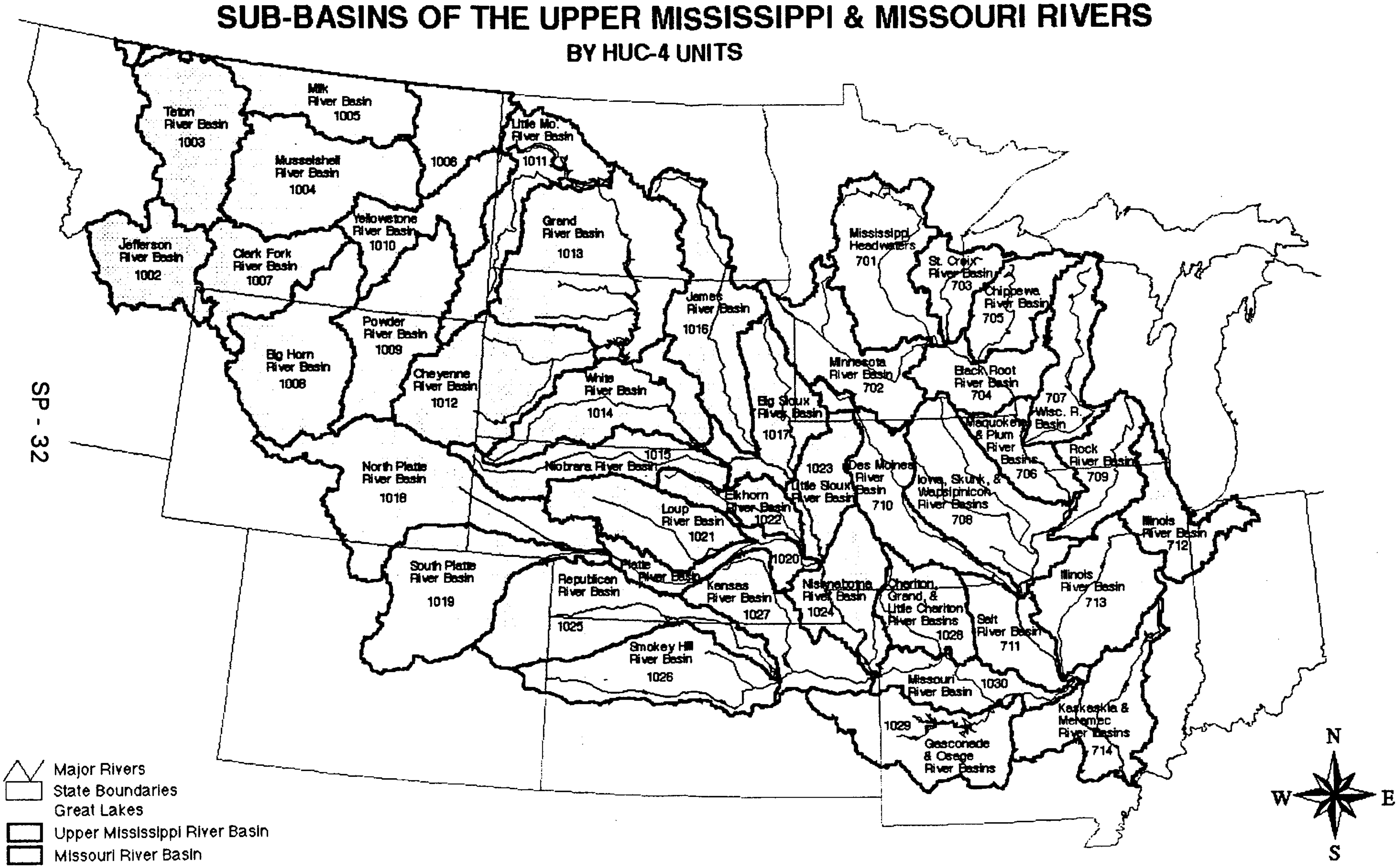
SUB-BASINS OF THE UPPER MISSISSIPPI & MISSOURI RIVERS

BY HUC-4 UNITS



FPMA

FloodPlain Management Assessment

Hydrology and Hydraulics

Rock Island District

May 1995
Final Report

US Army Corps
of Engineers

FLOODPLAIN MANAGEMENT ASSESSMENT
U.S. Army Corps of Engineers
Rock Island District

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STUDY PURPOSE

Following the 1993 flood, Congress tasked the Secretary of the Army to conduct a system wide study of floodplain management practices and policies of the Upper Mississippi and Lower Missouri Rivers and their tributaries. The study area crosses three Corps of Engineer Division boundaries and five District boundaries. Participating districts include St. Paul, Rock Island, and St. Louis along the Mississippi River, and Omaha and Kansas City along the Missouri River.

In order to accomplish the study objectives set forth by Congress, a systemic unsteady flow model of the Upper Mississippi and Lower Missouri Rivers was developed. Each district developed independent models which produced results that were assimilated by adjacent districts so that floodplain alternatives could be evaluated systemically. In addition to unsteady flow modeling of the major rivers, various upland watershed measures such as reservoirs and land treatments were evaluated to assess their potential impact on the 1993 flood.

BASIN DESCRIPTIONS

The following paragraphs describe basin characteristics for the Mississippi River and major tributaries within the Rock Island District.

Mississippi River

The drainage area of the Mississippi River within the Rock Island District ranges from 79,200 square miles at its upstream boundary near Guttenberg, Iowa, to 137,500 square, at Saverton, Missouri, its downstream boundary. The length of this reach is 314.0 miles. The average slope of the river along this reach is 0.5 feet per mile except in the vicinity of Rock Island, Illinois where prior to construction of the navigation locks and dams, the slope of the river was 1.5 feet per mile. Topography is generally characterized by high bluffs and rolling hills which descend to a wide, flat, floodplain adjacent to the river. Many small ungaged tributary streams as well as major rivers flow into the river along this reach. Plate RI-1 is a boundary map of the Rock Island District.

Rock River

Headwaters of the Rock River originate in the lake region of Fond du Lac County in Southeastern Wisconsin. The general direction of flow is south-southwest to the confluence with the Mississippi River at river mile (R.M.) 479.1 below Rock Island,

Illinois. The drainage area is 10,700 square miles at the mouth. The topography varies from flat and gently rolling farm land to steep uncultivated forest.

Iowa River

The Iowa River has a drainage area of 12,640 square miles, of which 7,870 square miles are contributed by its major tributary, the Cedar River. The average slope of the Iowa River is 1.9 feet per mile while the Cedar is slightly steeper with an average slope of 2.6 feet per mile. Both basins are long and narrow and roughly parallel one another with flow following a southeast path. The Cedar River joins the Iowa River 29.6 miles upstream from the confluence with the Mississippi River at Columbus Junction, Iowa. The Iowa River enters the Mississippi River at R.M. 434.4. Both basins are characterized by gently rolling topography and well drained soils.

Skunk River

The Skunk River basin extends from the north central region of Iowa to the Mississippi River in the southeast. The drainage area is 4,355 square miles. The topography within the basin is gently rolling with elevations ranging between 518 feet and 1,200 feet. Land use within the basin land is nearly all agricultural. The general shape of the basin is long and narrow with a length of 180 miles and an average width of 24 miles. The Skunk River splits into two main channels in Keokuk County forming the North Skunk and the South Skunk rivers. Below the junction, the river meanders through a narrow floodplain entering the Mississippi River at R.M. 396.0 in Lee County.

Des Moines River

The Des Moines River basin extends across central Iowa to the southeastern part of the state. The watershed has an area of 14,802 square miles. Des Moines, Ottumwa and Fort Dodge are the largest population centers. This basin contains 9.4 million acres of land and 42,000 acres of lake surface. The Des Moines River has numerous tributaries, many of which are very short with small drainage areas. Its major tributary is the Raccoon River which enters the Des Moines River in the city of Des Moines.

The Des Moines River basin has an average width of 40 miles and extends 360 miles from its headwaters north of Slayton, Minn., to its confluence with the Mississippi River below Keokuk, Iowa at R.M. 361.3. Downstream of its confluence with the Raccoon River near Des Moines, Iowa, the river valley changes both in direction and character. North of this point, the valley topography is shallow, with steep walls and a narrow flood plain. South of this point the river flows southeasterly in a broader, more mature flood plain in which the valley becomes wider and deeper with rounded bluffs.

Water Resources Development

As one of five districts that make up the North Central Division, the Rock Island District is responsible for administering federal water resource development programs in large portions of Iowa and Illinois and smaller portions of Wisconsin, Missouri and Minnesota.

The District operates and maintains 12 locks and dams on its portion of the mainstem Mississippi River and 8 locks and dams on the Illinois Waterway. It constructed and operates three flood control reservoirs in Iowa: Coralville Lake on the Iowa River, and Red Rock and Saylorville lakes on the Des Moines River.

The Coralville Lake Project is located on the Iowa River upstream from Iowa City in Johnson County and is a part of the general comprehensive plan for flood control and other purposes in the Upper Mississippi River basin. Construction began on this project in July 1949, and it was completed and put into operation in October 1958. The dam controls runoff from 3,115 square miles and provides protection to downstream reaches including the operation for the Mississippi River flood stages. The normal conservation pool at the dam is 683.0 feet National Geodetic Vertical Datum (NGVD) with 42,200 acre-feet of storage. The flood control (elevation 712.0 feet) has the capacity to store an additional 419,000 acre-feet or 2.62 inches of inches of runoff over the basin. The cumulative damages prevented since the project has been in operation (1959 through September 1993) are estimated at $49.2 million.

The Red Rock Dam and the Lake Red Rock Project on the Des Moines River are chiefly in Marion County, but extend into Jasper, Warren and Polk Counties. The dam is approximately 60 miles downstream from the City of Des Moines. The drainage area above the dam site is 12,323 square miles. A permanent lake of 265,500 acre feet storage area is formed behind the dam. With the flood control pool full (elevation 780.0 feet), the reservoir storage is 1,484,900 acre feet or 2.26 inches of runoff above the conservation pool of 742 feet NGVD. The net cumulative damage prevented since the project has been in operation (1969 through September 1993) is estimated at $390.4 million. Flood protection is provided to 36,000 acres of agricultural lands in the Des Moines River basin and to the Cities and Towns of Ottumwa, Eldon, Eddyville, Keosauqua and Farmington.

In 1958, Congress authorized construction of Saylorville Lake on the Des Moines River about 11 miles upstream from the City of Des Moines. The principal purpose of the Saylorville Project is to furnish needed additional storage to supplement the flood control capacity of the downstream Red Rock Dam and Lake Red Rock and to provide flood protection to the City of Des Moines. The permanent conservation pool forms a lake with storage of about 90,000 acre-feet and extends some 17 miles upstream from the dam. The reservoir has a total capacity of 676,000 acre-feet or 1.89 inches of runoff over the 5,823 square mile drainage area above the dam at full flood control pool elevation 890 feet and covers about 16,700 acres. The conservation pool was raised from 833 to 836 feet in 1983 to provide a water supply for the City of Des Moines and the Iowa Southern Utilities

near Ottumwa, Iowa. The Saylorville Project has been in operation since April 1977. Estimated damages prevented (from 1977 to 1993) are $156.3 million.

Along the Mississippi River, downstream from the mouth of the Des Moines River, levee districts and the cities of Quincy, Illinois and Canton, LaGrange, and Hannibal, Missouri also benefit from the combined operation of these three reservoirs.

Examples of other local flood control protection projects built by the Rock Island District include Waterloo, Evansdale, Des Moines, Clinton, Dubuque, Marengo, Marshalltown and Bettendorf, Iowa; Rockford, Fulton, East Moline, Rock Island, Milan, Illinois, and Hannibal, Missouri.

HYDROMETEOROLOGY

There are a number of factors that can create the conditions necessary for a major flood. These include: high base streamflows, heavy snow cover, heavy rainfall and saturated soil conditions throughout a watershed. All of these were present throughout the Rock Island District as the winter of 1992-93 gave way to spring.

Antecedent Conditions

The Upper Mississippi River basin including major tributary basins within the Rock Island District set near records for rainfall during the spring and summer of 1993. Although records were not broken in the fall of 1992, November and December were well above average. Particularly in November, rainfall totals were 2 to 3 times the average amount. Plate RI-2 depicts the deviation from normal precipitation for the period November 1982 through April 1993.

Description of Storms

The severe flooding that occurred in the Upper Mississippi River basin was the culmination of the wet spring and a series of storms in March, June and July. Flooding was influenced by not only the wet antecedent conditions and large rainfall totals, but also by the distribution of daily rainfall in July. The storm of July 4-5 was a significant event that produced a large amount of rainfall over southern Iowa. The conditions that existed in the mid-west which were responsible for this storm are described below in detail.

A high pressure system was positioned over the southeastern United States while an area of warm moist air flowed into the midwest as a result of the circulation around the southeastern high and the low pressure system in southern Canada. This pattern was typical for most of June and July. However, to aggravate the situation, a stationary front extended from northern Missouri across southeastern Iowa to southern Wisconsin, and a series of cold fronts rotated cyclonically around the low into western Iowa. The dense air behind each cold front collided into the warm air over Iowa, southern Minnesota and

southern Wisconsin. Aiding the production of rain was a strong area in the middle atmosphere that caused an additional upward motion. This produced additional lift to enhance the creation of thunderstorms. This area began over Kansas and Nebraska to the west of a cold front in western Iowa and proceeded over Iowa from the southwest. The jet stream that passed over Iowa from southwest to northeast also aided the production of rain and created what is called the chimney effect (strong winds in the upper atmosphere blow across the region of thunderstorm development). The effect of the winds was to evacuate air from the top of the thunderstorms, creating an updraft. More warm moist air was then drawn up into the thunderstorms and produced an exceptional amount of rain. As the storm moved northeastward at about 10 to 20 miles per hour, new thunderstorms formed to the southwest of the original thunderstorms and passed over the same areas, adding to the large rainfall totals. This storm produced a total of four to eight inches of rain across a 250-mile long path from Taylor County in southwest Iowa, northeastward through Oskaloosa, Marengo, Cedar Rapids and Dubuque.

Strong thunderstorms moved into central Iowa before sunrise on July 8 and rapidly traversed eastward across Iowa and into Illinois. A second set of thunderstorms developed over west central Iowa later that afternoon and slowly moved along the same path as the morning storms. By the time these storms weakened on July 9, a wide area of 3 to 9 inches of rain fell in an uninterrupted 275-mile long band from the Nebraska border at Onawam eastward through Denison, Ames, Marshalltown, Waterloo, Independence and Guttenburg, Iowa.

DESCRIPTION OF FLOODING

The Great Flood of 1993 was unique in its areal extent as well as its duration. It encompassed several months of relatively heavy rainfall that occurred at a time when the ambient conditions already posed an increased probability for flooding. Major tributaries as well as the main stem Mississippi River experienced new floods of record.

Mississippi River

The flooding on the Mississippi River was the most devastating in terms of property loss, disrupted businesses and personal trauma of any flood in the history of the United States. Millions of acres of farmland were under water for weeks during the growing season. Damaged highways and roads disrupted overland transportation throughout the flooded region. The river was closed to navigation for several weeks. Erosion of the banks and channel of the Mississippi River were noted in many reaches. In addition to erosion of the shoreline, loss of valuable topsoil was also a major problem. The extent and duration of the flooding caused numerous levees to fail. Every gaging station on the Mississippi below Lock and Dam 15 experienced a new flood of record. Above Lock and Dam 15, the 1993 Flood was surpassed by only one other event. A summary of the top five floods of record for the gaging stations along the Mississippi River main stem from Lock and Dam 11 downstream to Lock and Dam 22, is shown in Table RI-1.

Table RI-1

Mississippi River Top 5 Floods of Record

Station	River Mile above Ohio River	Flood Stage Feet	Date 1	Stage 1	Date 2	Stage 2	Date 3	Stage 3	Date 4	Stage 4	Date 5	Stage 5
Cassville	606.3	18.00	4/25/65	24.12	6/30/93	21.01	4/23/69	20.52	4/21/51	20.30	4/24/52	20.18
Dam 11	583	13.80	4/26/65	25.69	7/01/93	22.32	4/23/69	21.72	4/22/51	21.63	4/25/52	21.57
Dubuque	579.3	17.00	4/26/65	26.81	7/01/93	23.84	4/24/69	23.09	5/06/75	22.78	4/25/52	22.70
Dam 12	556.7	NA	4/26/65	23.51	7/01/93	21.50	4/26/52	20.11	4/24/69	20.03	4/23/51	19.98
Sabula	535	16.00	4/27/65	22.90	7/07/93	21.30	4/25/69	20.00	4/27/52	19.90	4/26/51	19.60
Dam 13	522.5	16.00	4/28/65	25.03	7/08/93	22.17	4/26/69	21.37	4/27/52	21.23	4/26/51	21.00
Clinton	518	16.00	4/28/65	24.85	7/08/93	22.98	4/26/69	21.25	4/27/52	20.90	4/26/51	20.70
Camanche	512	17.00	4/28/65	24.65	7/08/93	22.98	4/26/69	21.53	4/27/52	21.20	10/6/86	20.85
Princeton	502	13.00	4/28/65	20.10	7/08/93	18.28	4/26/69	16.90	4/27/52	16.80	4/26/51	16.60
Dam 14	493.3	11.00	4/28/65	17.75	7/08/93	16.56	4/26/69	14.56	5/08/75	14.03	4/28/52	14.01
Dam 15	482.9	15.00	7/09/93	22.63	4/28/65	22.48	4/26/69	19.30	6/17 1892	19.30	10/7/86	19.22
Fairport	463.1	14.00	7/09/93	24.74	4/28/65	23.60	4/26/93	20.20	4/26/73	20.11	4/26/69	20.00
Dam 16	457.2	13.80	7/09/93	24.01	4/27/65	23.27	4/25/73	20.05	4/25/93	19.70	4/26/69	19.69
Muscatine	455.2	16.00	7/09/93	25.61	4/29/65	24.81	4/25/73	21.63	4/25/93	21.30	4/26/69	21.20
Dam 17	437.0	17.43	7/09/93	25.90	4/28/65	23.14	4/25/73	21.73	4/24/93	21.04	4/26/69	19.74
Keithsburg	428.0	13.00	7/09/93	24.15	4/27/65	20.36	4/22/93	19.66	4/25/73	19.35	10/7/86	17.46
Oquawka	415.9	15.00	7/10/93	28.30	4/28/65	24.20	4/25/73	24.02	4/24/93	23.50	10/7/86	21.90
Dam 18	410.5	9.00	7/10/93	21.54	4/25/73	17.90	4/27/65	17.10	4/30/93	16.58	10/6/86	14.72
Burlington	403.1	15.00	7/10/93	24.98	4/25/73	21.50	4/10/65	21.00	4/25/93	20.40	10/6/86	19.02
Dam 19 Keokuk	364.2	16.00	7/10/93	27.58	4/23/73	23.50	5/01/65	22.14	4/04/60	21.64	5/28/44	20.60
Gregory Landing	352.9	15.00	7/09/93	28.49	4/24/73	24.58	5/01/65	22.71	4/03/60	21.50	4/22/93	21.46
Dam 20	343.2	14.00	7/09/93	27.88	4/23/73	24.50	5/01/65	21.42	10/4/86	20.92	4/22/93	20.45
La Grange	336.0	16.00	7/13/93	28.30	4/25/73	25.30	4/27/93	23.35	5/1/65	22.40	10/4/86	22.20
Quincy	327.0	17.00	7/13/93	32.13	4/23/73	28.90	10/4/86	25.33	4/28/65	24.80	4/04/60	24.30
Dam 21	324.9	17.00	7/13/93	31.30	4/25/73	27.70	10/4/86	24.86	4/28/65	23.89	4/29/93	23.70
Hannibal	309.9	16.00	7/16/93	31.80	4/25/73	28.59	10/4/86	24.85	5/01/65	24.59	4/26/93	24.16
Dam 22	301.2	14.60	7/25/93	29.58	4/25/73	26.80	10/5/86	24.10	4/28/93	23.51	4/05/83	23.40

Iowa River

Flooding in the Iowa River and Cedar River basins began in late March and early April, with near record flooding on the Cedar River at Cedar Rapids. Torrential rains continued during the summer of 1993 and the Cities of Cedar Rapids, Marshalltown, Waterloo, Conesville and Wapello experienced additional major flooding. Many of these areas were flooded in July and again in August as the relentless rains continued to cross Iowa. There was some flood damage reduction due to the local flood protection projects as well as to the operation of Coralville Dam.

Des Moines River

The flooding on the Des Moines River had the greatest impact on the City of Des Moines. The record rainfall that persisted in the basin resulted in overtopping of some levees and inundation of the city's water treatment plant. Residents were without drinking water for three weeks. The storm responsible for this event began at 9 p.m. on July 8 and ended on July 9 at 1 a.m. During that five-hour period, 6.5 inches of rain fell on an already saturated Des Moines River basin. Flash flooding occurred on the Raccoon River, a tributary to the Des Moines River that has its confluence in the City of Des Moines. This compounded the severity of the flooding in Des Moines. The operation of the two reservoirs that are located on the Des Moines River and the impact that they had on the flooding are discussed in the section on upland measures.

ANALYSIS OF AGRICULTURAL LEVEES

Many questions have been raised following the 1993 flood concerning the impact levees have on flood heights. Evaluation of levee action alternatives focuses on agricultural levees because the vast amount of land protected by these levees offers the potential for storage of flood waters. In most cases, limited opportunity for storage or conveyance of flood water exists behind urban levees because of the relative size of the protected area. Plate RI-3, Index Map, shows the general location of both urban and agricultural levee projects along the Mississippi River within the Rock Island District.

Methodology

The unsteady flow model UNET was selected to assess the hydraulic effects of agricultural levees along the Mississippi River in the Rock Island District. The program, developed by Dr. Robert Barkau, simulates one dimensional unsteady flow in a network of open channels. Modeling of alternatives was performed on a system wide basis which encompassed four Corps districts, Rock Island and St Loius on the Mississippi River, and Omaha and Kansas City on the Missouri River. In order to accomplish this, overlap between adjacent Corps districts had to be included in upstream models. Iterative simulations were then performed to transfer results between the adjacent districts for the overlap portion of the model. Further simulations were considered unnecessary when computed stages at the transfer point in the overlap area common to both models agreed

within 0.1 feet. Grafton, Illinois, was selected as the location to transfer results from Rock Island to St. Louis.

Flood Frequency

Because of the inherent complexity involved in developing frequency based water surface profiles within the short time frame of this assessment, the 1993 flood was chosen as the base condition for comparing levee alternatives.

The Mississippi River flow frequency estimates from which published water surface profiles were developed in 1979, used flow records through 1975. In that analysis, mean and standard deviations were plotted against respective drainage areas and were fitted with smoothed curves. These smoothed statisics were used to develop flow frequency estimates for various gages located along the Mississippi River.

A preliminary statistical analysis was performed updating streamflow records through 1993 for Mississippi River gages operated by the US Geological Survey at Clinton and Keokuk, Iowa. These statistics were plotted against the regional curves developed in 1979. Comparison of the new values with regional curves show that a change in the regional curves is not warranted at this time. Until a more detailed study is funded and completed, the published 1979 flow frequency values will continue to be used.

The peak discharge observed at Clinton during 1993 was 239,000 cfs which occurred on July 7. That discharge falls in the range of a 25 to 50 year event. At Keokuk, a peak discharge of 446,000 cfs occurred on July 10. This is greater than a 500 year event which illustrates the wide variation in frequency associated with the 1993 flood within the Rock Island District.

Model Extent

The Rock Island District model extends form Lock and Dam 10 at Guttenburg, Iowa, R.M. 615.0, to R.M. 301.0 at Lock and Dam 22 near Saverton, Missouri. Modeling of levee alternatives was concentrated in the reach between Lock and Dam 16, R.M. 460.0 to R.M. 301.0. Above R.M. 460.0, only urban levees and one or two small agricultural levee systems exist as the floodplain is much narrower. Gaged tributaries are also modeled upstream to the first gaging site above the mouth. A schematic of the Rock Island model showing upper and lower boundaries and tributary inputs to the model is shown on plate RI-4. Table RI-2 lists the location and drainage area at each lock and dam in the Rock Island District along the Mississippi River.

Table RI-2

Mississippi River
Navigation Locks and Dams
Within the Rock Island District

Name	Location (River Mile)	Drainage Area (Sq. Mi.)
Lock and Dam No. 11	583.0	82,100
Lock and Dam No. 12	556.7	82,500
Lock and Dam No. 13	522.5	85,600
Lock and Dam No. 14	493.3	88,400
Lock and Dam No. 15	482.9	88,500
Lock and Dam No. 16	457.2	99,500
Lock and Dam No. 17	437.0	99,600
Lock and Dam No. 18	410.5	113,600
Lock and Dam No. 19	364.2	119,600
Lock and Dam No. 20	343.2	134,300
Lock and Dam No. 21	324.9	135,200
Lock and Dam No. 22	301.2	137,500

Model Geometry

Model geometry was compiled from two sources. The main stem channel geometry was developed from channel surveys conducted for the purpose of channel maintenance. Elevations are recorded to the nearest 0.1 feet. Overbank geometry was developed from USGS 7.5 minute quadrangle maps with a contour interval of 10 feet. Cross sections extend from bluff to bluff and are spaced at intervals of 0.5 to 2.0 miles along the channel. Tributary cross sections were also developed from USGS quad maps with the channel geometry defined by soundings collected at USGS gages during stream flow measurements. Each tributary and mainstem reach between tributary junctions, serves as a routing reach in the model. The model schematic on plate RI-4 shows reach locations.

Channel Roughness

The primary means of adjusting conveyance in the UNET model is Manning's "n" coefficient which governs the frictional force exerted by the boundary of the channel on the flowing water. For large rivers, Manning's "n" varies with depth, because the relative impact of roughness features, such as dunes in the river bed, the height of vegetation, etc., decreases with increasing depth. The goal of the calibration process is to determine the variation of Manning's "n" with depth. The automatic calibration option within the UNET program adjusts conveyance at cross-sections between gaging stations such that the model reproduces observed stages at gaging stations at the upper end of a reach under steady

flow conditions. The automatic calibration routine is explained in the section on model calibration.

Inflow Data

Inflow to the model consists of tributary inflows and local inflows representing ungaged drainage area. For the Rock Island model, discharges from Lock and Dam 10 served as the upstream boundary on the mainstem. All tributary inflows were from continuous record USGS gaging stations. Local inflows were developed from discharges at index gages and proportioned by drainage area with the flows being distributed uniformly along the reach of interest.

Model Calibration

Model calibration was accomplished using an automated calibration technique whereby channel conveyance, i.e. roughness, is adjusted to provide a best fit rating curve through a scatter plot of observed stages versus computed discharges at mainstem gaging stations. Two floods were simulated for the purpose of model calibration. The 1986 flood was chosen to represent an event in which no levees were breached. The 1993 flood was calibrated to reproduce an event in which levees overtopped. As mentioned above, computed stages for the 1993 event serve as the basis for assessment of agricultural levee alternatives. Plates RI-5 through RI-8 show computed vs. observed stage hydrographs for the 1993 flood at Muscatine, Iowa, Burlington, Iowa, Quincy, Illinois, and Hannibal, Missouri. Table RI-3 compares 1993 computed peak water surface elevations with observed elevations at Mississippi River gages located between Lock and Dam 10 and Lock and Dam 22 at Saverton, Missouri.

Table RI-3
1993 Computed vs. Observed Maximum Water Surface Elevations

Gage	River Mile	Computed WSEL MSL	Observed WSEL MSL	Difference in Feet
L&D 10 TW	615.1	620.1	620.1	0.0
CASSVILLE	606.3	617.2	617.3	-0.1
SPECHTS FERRY	592.3	612.4	612.3	+0.1
L&D 11 TAIL	583.0	610.7	610.5	+0.2
DUBUQUE	579.3	609.4	609.3	+0.1
L&D 12 TAIL	556.7	602.1	601.7	+0.4
SABULA	535.0	593.3	593.6	-0.3
L&D 13 TAIL	522.5	591.0	590.9	+0.1
CLINTON	518.0	588.7	589.3	-0.6
CAMANCHE	511.8	586.5	586.2	+0.3
PRINCETON	502.1	581.8	581.8	0.0
L&D 14 TAIL	493.3	573.0	573.6	-0.6
48TH ST. MOLINE	487.9	570.2	569.6	+0.6
L&D 15 TAIL	482.9	565.1	565.1	0.0
FAIRPORT	463.5	559.5	559.9	-0.4
L&D 16 TW	457.2	557.0	557.9	-0.9
MUSCATINE	455.2	556.0	556.3	-0.3
L&D 17 TAIL	437.1	551.7	552.4	-0.7
KEITHSBURG	428.0	546.7	547.3	-0.6
OQUAWKA	415.9	541.5	541.5	0.0
L&D 18 TAIL	410.5	540.1	540.1	0.0
BURLINGTON	403.1	536.4	536.4	0.0
FORT MADISON	383.9	528.0	528.4	-0.4
L&D 19 TAIL	364.3	505.3	505.4	-0.1
GREGORY LANDING	352.9	500.3	501.2	-0.9
L&D 20 TAIL	343.2	495.8	496.4	-0.6
LAGRANGE	336.0	492.8	492.9	-0.1
QUINCY	327.9	490.0	490.7	-0.7
L&D 21 TAIL	324.9	488.8	489.1	-0.3
HANNIBAL	309.9	481.4	481.2	0.2
L&D 22 TW	301.2	476.0	475.7	0.3

As shown, computed peak water surface elevations were within one foot of observed values at all mainstem gages. Tributaries were included in the model to provide the timing effects of inflow but were not calibrated to the degree of accuracy of the mainstem because of the lack of detailed geometry. Model calibration was performed by Dr. Barkau under contract with the Rock Island District.

Levee Simulation

There are 19 agricultural levee districts along the Mississippi River within the Rock Island District. During the 1993 flood, 14 of these agricultural levees were overtopped as the water level in the river exceeded the design height of these levees by several feet. In order

to simulate overtopping of a levee, areas behind the levees are treated as storage cells which act as lakes with a horizontal water surface. Only storage is considered for alternatives in which the levees remain intact. Storage volume within each cell is defined by the cell area and the height of the levee above the natural ground.

Two options are available for simulating the overtopping of levees. If an observed event is being simulated and the time the levee failed is known, that time can be specified in the model. If that time is unknown, then the relationship between the water surface elevation of the river and the elevation of the levee crown governs when the levee overtops.

LEVEE ALTERNATIVES

Four levee alternatives were assessed on a system wide basis to determine what if any impact they would have had on the 1993 flood had they been implemented prior to the event. The alternatives are: 1) levees raised to an infinite height allowing containment of the 1993 flood, 2) levees with a weir section constructed to a 25-year design height providing a uniform level of protection throughout the system, 3) removal of all levees that primarily protect agricultural land, and 4) an alternative in which all levees are set back a distance equal to 50% of the existing floodway width. A fifth alternative in which the addition of flood fight freeboard would be prohibited, was analyzed as a separate case study by the Rock Island District. Water surface elevations corresponding to each alternative will be compared to the computed water surface elevations from the existing condition calibration simulation of the 1993 flood. Table RI-4 shows the peak stage difference between 1993 computed water surface elevations and each of the levee alternatives modeled. Table RI-5 shows the percentage change in peak discharge between levee alternatives and 1993 computed discharges. Table RI-6 lists levee data for each drainage district and whether the levee was overtopped during the simulation of each alternative. Plates RI-9 through RI-24 compare stage hydrographs generated from UNET simulations of each alternative with 1993 computed flood hydrographs at Muscatine, Iowa; Burlington, Iowa; Quincy, Illinois; and Hannibal, Missouri. Inundation maps of the 1993 base condition and levee action alternatives in the lower portion of the Rock Island District, are shown on plates RI-25 through RI-30.

Table RI-4.

1993 Peak Stage Difference for Agricultural Levee Alternatives

Gage	River Mile	Computed WSEL MSL	Agricultural No Levee (ft)	Natural No Levee (ft)	1993 Confined (ft)	Limited Floodfight (ft)	25 Year (ft)	Setback 1993 (ft)	Setback No Fail (ft)
L&D 16 TW	457.2	557.0	-5.7	-3.7	0.3	0.0	-0.5	-1.5	-1.4
MUSCATINE	455.2	556.0	-5.9	-3.9	0.3	0.0	-0.6	-1.5	-1.4
BASS ISLAND	448.4	554.0	-6.5	-3.5	0.3	0.0	-0.7	-1.6	-1.4
PORT LOUISA	441.3	552.8	-5.9	-2.8	0.4	0.0	-0.9	-1.6	-1.3
L&D 17 TAIL	437.1	551.7	-5.4	-2.4	0.4	0.0	-0.9	-1.8	-1.4
KEITHSBURG	428.0	546.7	-4.9	-2.2	0.3	-0.1	-0.7	-1.6	-1.2
OQUAWKA	415.9	541.5	-4.2	-2.0	0.5	-0.5	-1.1	-1.4	-0.9
L&D 18 TAIL	410.5	540.1	-3.8	-1.8	0.4	-0.7	-1.2	-1.5	-1.0
BURLINGTON	403.1	536.4	-1.7	-0.8	0.3	-0.8	-1.1	-1.0	-0.6
DALLAS CITY	390.7	531.0	-0.8	-0.5	0.2	-0.7	-0.8	-0.5	-0.2
FORT MADISON	383.9	528.0	0.0	-0.1	0.2	-0.5	-0.6	-0.1	0.2
L&D 19 TAIL	364.3	505.3	-4.6	-1.1	2.5	-1.0	-1.1	-0.4	0.6
WARSAW	359.9	503.6	-5.8	-1.5	2.4	-1.2	-1.2	-0.6	0.4
GREGORY LANDING	352.9	500.3	-6.2	-1.3	2.3	-0.8	-0.7	-0.4	0.7
L&D 20 TAIL	343.2	495.8	-4.9	-0.7	3.5	-0.8	-0.6	-1.2	0.9
LAGRANGE	336.0	492.8	-4.2	-0.2	3.4	-1.4	-1.2	-1.3	1.0
QUINCY	327.9	490.0	-3.4	0.4	3.8	-2.7	-2.3	-1.2	1.6
L&D 21 TAIL	324.9	488.8	-3.4	0.6	4.0	-3.0	-2.8	-1.4	1.7
HANNIBAL	309.9	481.4	-5.8	0.6	4.2	-3.2	-3.3	-0.7	2.8
L&D 22 TW	301.2	476.0	-5.1	-1.0	3.5	-2.9	-3.3	-0.6	2.2

Note: Values in this table are approximate and appropriate only for this assessment. A more detailed model is required to accurately estimate the flow capacity of the floodplain. Roughness values for the floodplain were selected to represent variations in land use and provide an upper and lower bound for overbank conveyance. Data on bridges, roads, railroad embankments, and existing vegetation were unavailable for the model. As a result, effective overbank flow area is overstated at some locations. Although further analysis may result in different stages for the without levee conditions, the general trends should remain the same.

Table RI-5.

Percentage Difference in 1993 Peak Discharges for Agricultural Levee Alternatives

Gage	River Mile	Computed Discharge cfs	Agricultural No Levee	Natural No Levee	1993 Confined	Limited Floodfight	25 Year	Setback 1993	Setback No Fail
L&D 16 TW	457.2	282,300	+1.3	+1.1	+0.6	-0.4	-0.6	+0.4	+0.4
MUSCATINE	455.2	282,300	+1.3	+1.0	+0.6	-0.2	-0.4	+0.4	+0.4
BASS ISLAND	448.4	282,400	+1.1	+0.7	+0.7	+0.6	-1.3	+0.7	+0.5
PORT LOUISA	441.3	284,300	+1.0	+0.7	+1.0	+0.9	-1.4	+0.8	+0.7
L&D 17 TAIL	437.1	283,000	+2.4	+2.0	+2.1	+1.0	-1.6	+1.8	+1.8
KEITHSBURG	428.0	386,700	+2.4	+1.3	+2.9	+1.0	-4.1	+0.8	+3.0
OQUAWKA	415.9	391,400	+1.0	-0.7	+3.1	+1.3	-5.8	+0.1	+2.9
L&D 18 TAIL	410.5	392,300	+0.0	-1.5	+3.2	+1.4	-6.0	-0.1	+3.0
BURLINGTON	403.1	392,200	-0.2	-1.9	+3.2	-3.7	-6.4	-0.1	+2.9
DALLAS CITY	390.7	435,500	-0.3	-1.3	+2.2	-5.3	-6.8	-0.5	+2.1
FORT MADISON	383.9	435,800	-0.1	-1.3	+2.3	-5.2	-6.6	-0.6	+2.1
L&D 19 TAIL	364.3	436,600	+0.2	-1.0	+2.4	-4.5	-6.0	-0.6	+2.1
GREGORY LANDING	352.9	492,200	+11.5	+10.5	+11.9	-9.1	-7.4	+7.8	+12.0
L&D 20 TAIL	343.2	489,200	+11.5	+9.6	+12.6	-9.4	-8.1	+7.0	+12.5
LAGRANGE	336.0	517,100	+7.2	+5.4	+8.1	+2.0	+0.6	+4.2	+8.2
QUINCY	327.9	515,400	+7.5	+4.9	+8.3	-0.7	+0.7	-0.4	+8.2
L&D 21 TAIL	324.9	511,300	+8.4	+5.6	+9.2	+0.1	+1.5	+0.4	+9.1
HANNIBAL	309.9	501,100	+15.4	+11.5	+16.3	-13.7	-11.9	+6.6	+15.8
L&D 22 TW	301.2	499,400	+15.2	+11.2	+16.7	-14.0	-12.1	-4.3	+16.4

Table RI-6
Levee Failure Data

Levee District	River Mile	Acres Protected	1993 Computed	Limted Floodfight	25-YR Notch	No Reservoir	5% Runoff Reduction	10% Runoff Reduction	Setback 1993 Elevation
DRURY	459.0-451.0	5050	NO	NO	YES	NO	NO	NO	NO
BAY ISLAND	450.9-434.2	24630	NO	NO	YES	NO	NO	NO	NO
DES MOINES #4	433.5-422.3	22470	NO	NO	YES	NO	NO	NO	NO
DES MOINES #7	422.1-410.2	19960	NO	NO	YES	NO	NO	NO	NO
DES MOINES #8	410.2-406.0	3650	NO	YES	YES	NO	NO	NO	NO
HENDERSON #3	414.8-411.8	2380	YES	YES	YES	YES	NO	NO	NO
HENDERSON #1	412.3-403.2	8330	NO	YES	YES	NO	NO	NO	NO
HENDERSON #2	403.2-400.8	7870	NO	YES	YES	NO	NO	NO	NO
GREEN BAY	396.0-386.6	13690	YES	YES	YES	YES	NO	NO	YES
DES MOINES-MISS	359.8-358.6	12710	YES	NO	YES	YES	YES	YES	YES
MISS-FOX	358.6-354.3	11030	YES	YES	YES	YES	YES	YES	YES
HUNT-LIMA	358.6-341.7	31080	YES	YES	YES	YES	YES	NO	YES
GREGORY	354.4-347.8	9270	YES	YES	YES	YES	YES	YES	YES
INDIAN GRAVE UPPER	341.7-336.2	12400	YES	YES	YES	YES	YES	NO	NO
INDIAN GRAVE LOWER	335.6-330.0	6810	YES	YES	YES	YES	YES	YES	YES
UNION TOWNSHIP	335.3-331.5	3860	YES	YES	YES	YES	YES	YES	YES
FABIUS	332.4-323.5	14960	YES	YES	YES	YES	NO	NO	NO
MARION CO.	323.5-320.7	4170	YES	YES	YES	YES	YES	NO	YES
SOUTH RIVER	320.5-312.1	10200	YES	YES	YES	YES	YES	NO	YES
SNY UPPER	315.4-296.8	42070	YES	YES	YES	YES	YES	YES	YES
SNY MIDDLE	296.0-275.0	58740	NO	YES	YES	NO	NO	NO	NO
SNY LOWER	272.0-266.0	10900	NO	NO	YES	NO	NO	NO	NO

Yes = Overtop

No fail levees for both existing and setback levee alignments were not included in the table since levees are prevented from overtopping in both alternatives.

Existing Condition

Levees in the existing condition model include freeboard added to the levee crown during flood fight operations. Although the additional freeboard did not in most cases prevent levee overtopping during the 1993 flood, it did affect the timing of the flows and stages. Most of the agricultural levees in the Rock Island District were designed to provide protection up to a 50-year frequency flood event. In the lower portion of the Rock Island District, the 1993 flood was in excess of a 500-year event. Had additional height not been added to the levees, overtopping would have occurred much earlier in the event. Overtopping of levees in the existing condition model were reproduced on the dates and times they actually occurred. In all other alternatives modeled, levee overtopping was dependent on the relationship between the levee crown elevation and the water surface elevation of the river. Timing of levee overtopping plays an important role in determining the effects levees have on flood stages. Levees which overtop close to the peak of the event, may have a substantial impact on flood stages. Discharge hydrographs, Plates RI-30 through RI-33, show how flow and timing are affected by raising or lowering levees.

25-Year Controlled Failure

The 25-year controlled failure alternative is modeled by setting the downstream end of a levee at an elevation equal to the 25-year frequency at that location. This would be done uniformly for all agricultural levees within the study reach. The intent is to minimize damage by causing flood water to back into the protected area at low velocity. In practice an overflow section would be provided at the selected location. Floodfighting would be prohibited.

As shown in table RI-4, reduction in 1993 flood stages are on the order of 0.5 to 1.0 feet from Muscatine, Iowa to La Grange, Missouri. Below La Grange, reductions were on the order of 2.0 to 3.5 feet. The reductions below La Grange were greater because of timing effects coupled with the failure of the middle and lower cells of the Sny Levee and Drainage District which in total protect 69,700 acres. Neither cell failed during the 1993 flood. Plates RI-9 through RI-12 compare stage hydrographs of the 1993 flood to hydrographs of the 25-year controlled overtopping alternative.

Levees Raised Above The 1993 Event

An alternative in which existing levees are raised to a height such that the 1993 flood would not exceed the top of the levees was modeled by making the levees infinitely high. As shown in table RI-4, above Lock and Dam 19, increases in river stages were minimal. However, downstream of Lock and Dam 19, river stages were 2.5 feet to 4.0 feet higher. This is attributed to the large number of levee systems at the downstream end of the Rock Island District. During the 1993 event, all of the levee systems downstream of Lock and Dam 19 were overtopped. Therefore, confining flow to the floodway between the levees raised the water surface profile since the effect of levee storage which was available during

the 1993 flood was lost. Plates RI-9 through RI-12 compare stage hydrographs of the 1993 flood to hydrographs of the confined floodway alternative.

Levees Removed

Removal of all primarily agricultural levees was simulated by removing encroachments and storage cell connections in the model making the entire overbank available for conveying flood waters. Two cases representative of varying land usage in the reclaimed floodplain were modeled for this alternative. Case 1, assumes that land use between the levee and the bluff is agricultural. This condition is represented by a 0.080 overbank "n" roughness coefficient. Case 2 assumes that this area would be allowed to revert to a natural or forested condition with trees and other vegetation being prominent. A 0.320 "n" value was selected to represent this condition. Land use between the river and the levee was assumed to remain the same as it is now.

Selection of roughness values for the no levee alternative was intended to provide an upper and lower bound for conveyance of overbank flow if the levees were to be removed. However, factors affecting conveyance were not evaluated in detail. As a result, the no levee model overstates the available effective flow area. For example, removal of the levee would not result in an effective flow width equal to the entire valley width. Physical factors such as channel meandering, vegetation, topography, structures such as roads and railroads, and other components will restrict effective flow width to a value much less than the cross section width used in model at some locations. Modifying the UNET model to accurately reflect the conveyance changes at every cross section was not feasible because of limited topographic data describing overbank features and time constraints. In an attempt to compensate for this deficiency, the Manning's N value representing agricultural land use was increased from 0.04 to 0.08 and from 0.16 for a natural wooded floodplain to 0.32. This adjustment is the same as reducing the overbank effective flow area by 50-percent. This adjustment does not reduce the area available for overbank flood storage. However, because of these assumptions, computed results for the levee removal alternative should be regarded as estimates. More precise and accurate simulation of this alternative would require the construction of an entirely new model and detailed studies to determine the long term effects of vegetation and sedimentation within the floodplain on conveyance. Table RI-4 shows that for agricultural land use, reduction in stages ranged from 0 to 6 feet. For a forested floodplain, changes ranged from an increase of 1 foot to a reduction of 4 feet. Plates RI-13 through RI-16 compare hydrographs of the 1993 flood to hydrographs generated by removing all agricultural levees.

Levees Setback

A levee setback alternative was modeled to determine the effects setting the levees back would have on 1993 peak stages. The setback distance used in simulating this alternative was 1.5 times the width of the existing floodway width or a minimum floodway of 5000 feet. For example if the floodway is currently 6000 feet wide at a particular location, with the levee setback, floodway width is increased to 9000 feet. Two setback alternatives

were examined. In one alternative levees would remain at their present design height plus freeboard added during the 1993 floodfight. The second alternative would raise the levees infinitely high removing the threat of overtopping.

In the case of levee setbacks the area between the location of the existing levee and the setback levee, becomes available for conveyance. Only storage effects are considered behind setback levees. Table RI-4 shows the difference in peak stage attributed to setting levees back from there existing location. For levees setback but built to the same height as they were during the 1993 flood, 1993 peak stages were reduced by as much as 2 feet. However, if the levees were setback but raised high enough to prevent overtopping during the 1993 flood, the change in peak stage varied from a reduction of 1.5 feet near Muscatine, Iowa to an increase of 3 feet near Hannibal, Missouri. Plates RI-17 through RI-20 compare stage hydrographs generated from setback levee alternatives with 1993 computed hydrographs.

Limited Floodfight

An alternative which limits floodfight activities to measures that maintain levee integrity without adding additional height to the levee, was modeled using the design levee crown as the overtopping elevation. Again, the effects were most noticeable at the downstream end of the study reach. Failure of the middle cell of the Sny Levee and Drainage District reduced stages below Quincy, Illinois by nearly 3 feet due to the timing of the failure in the simulation and the large area available for storage. Upstream of Keokuk, Iowa, reductions in stage due to limiting floodfight efforts were less than 1 foot since most of those levees did not overtop in either the limited floodfight simulation or during the actual 1993 event. See Table RI-4. Plates RI-21 through RI-24 compare stage hydrographs of the limited floodfight alternative with 1993 computed hydrographs.

UPLAND ALTERNATIVES

The following sections discuss upland flood damage reduction measures. As in the case of agricultural levee alternatives, performance of upland alternatives were evaluated by comparing each alternative with the 1993 flood, the base condition for this assessment. The following upland alternatives were investigated by the Rock Island District: the 1993 flood without existing reservoirs, increased retention at existing reservoirs, revised operation of existing reservoirs, additional reservoirs, and sensitivity of Mississippi River stages to upland runoff reduction measures.

NO RESERVOIRS

In order to determine the impact reservoirs had on the 1993 flood, hydraulic routings were performed using reconstituted hydrographs of the 1993 flood without reservoir holdouts from Coralville, Saylorville, and Red Rock. Tables RI-7 and RI-8 show the stage reductions attributed to the reservoirs along the Iowa and Des Moines Rivers on which those reservoirs are located.

Table RI-7

Stage Reductions on the Iowa River
Attributed to Coralville Reservoir

	Observed		Reconstituted		
Station	Stage (feet)	Flow (cfs)	Stage (feet)	Flow (cfs)	Reduction (feet)
Iowa City	28.5	28,000	30.1	39,000	1.6*
Lone Tree	22.9	57,000	23.9	65,000	1.0*
Wapello	29.5	121,000*	30.0	130,400	0.5*

*Estimated from Rating Curve

Table RI-8
Stage Reductions on the Des Moines River
Attributed to Saylorville and Red Rock Reservoirs

	Observed		Reconstituted		
Station	Stage (feet)	Flow (cfs)	Stage (feet)	Flow (cfs)	Reduction (feet)
2nd Ave	31.5	55,000	34.2	66,000	2.7
SE 14th St	34.3	116,000	34.7	122,000	0.4
Tracy	24.2	109,000	25.5	146,000	1.3
Ottumwa	22.1	112,000	23.9	140,000	1.8
Keosauqua	32.7	111,000	35.4	134,000	2.7
St.Francisville	32.0	120,000	33.1	138,000	1.1

The effect of reservoirs on Mississippi River stages was determined by using the reconstituted flows at Wapello and Keosauqua in the UNET model in lieu of the observed flows. This allowed the impact of reservoirs to be evaluated on a system wide basis. Results of the analysis are shown in table RI-9 in terms of increase in flood heights without reservoirs. Plates RI-34 through RI-37 compare stage hydrographs of the no reservoir condition with 1993 flood hydrographs for March through September at Muscatine, Burlington, Quincy, and Hannibal. Similarly, plates RI-38 through RI-41 show flow hydrographs for the same conditions.

Table RI-9

1993 Peak Stage Difference for Upland Alternatives

Gage	River Mile	Computed WSEL MSL	No Reservoirs (ft)	5% Runoff Reduction (ft)	10% Runoff Reduction (ft)
L&D 16 TW	457.2	557.0	0.0	-0.8	-1.6
MUSCATINE	455.2	556.0	0.0	-0.7	-1.5
BASS ISLAND	448.4	554.0	0.0	-0.8	-1.5
PORT LOUISA	441.3	552.8	0.0	-0.8	-1.6
L&D 17 TAIL	437.1	551.7	0.0	-0.8	-1.6
KEITHSBURG	428.0	546.7	0.0	-0.6	-1.2
OQUAWKA	415.9	541.5	0.0	-0.9	-1.6
L&D 18 TAIL	410.5	540.1	0.0	-1.0	-1.7
BURLINGTON	403.1	536.4	0.0	-0.9	-1.7
DALLAS CITY	390.7	531.0	0.0	-0.8	-1.4
FORT MADISON	383.9	528.0	0.0	-0.6	-1.1
L&D 19 TAIL	364.3	505.3	0.0	-0.5	-1.1
WARSAW	359.9	503.6	0.0	-0.6	-0.9
GREGORY LANDING	352.9	500.3	0.2	-0.4	-0.6
L&D 20 TAIL	343.2	495.8	0.8	-0.5	-1.3
LAGRANGE	336.0	492.8	0.4	-0.7	-1.3
QUINCY	327.9	490.0	0.3	-0.9	-1.4
L&D 21 TAIL	324.9	488.8	0.3	-1.0	-1.5
HANNIBAL	309.9	481.4	0.4	-1.0	-1.9
L&D 22 TW	301.2	476.0	0.3	-0.8	-1.5

Large rainfall events which occurred below the reservoirs in conjunction with the reservoirs being filled to capacity when peak flooding on the mainstem occurred, lessened the reductions in stage on the Mississippi River relative to those experienced on the tributaries.

In general stage reductions provided by the reservoirs along the tributaries were of vital importance in protecting property and lives in communities downstream. Without the reservoirs, levees protecting urban areas and critical facilities would have been in jeopardy.

INCREASED RESERVOIR RETENTION

All three major flood control reservoirs within the Rock Island District, Saylorville, Red Rock, and Coralville, were operated beyond full flood control capacity during the 1993 flood event. Deviations from approved regulation plans allowing lower than prescribed release rates were granted by higher authority in order to aid flood fight efforts in downstream communities and minimize impacts to affected critical facilities. As a result Saylorville and Red Rock Reservoirs on the Des Moines River rose to 2 to 3 feet above designated full flood control pool levels. Coralville Reservoir rose to nearly 5 feet above its full flood pool level. High pool levels begin to impact property and facilities upstream as well as raise concerns about dam safety. Peak pool stages at all three reservoirs were coincident with the real estate guide taking line. For these reasons, increased retention beyond the range described above would not be prudent without assessing the need to acquire additional real estate holdings and adequacy of remedial works upstream of the reservoirs.

REVISED RESERVOIR OPERATION

In order to assess whether any modification of reservoir operations could have lessened the impact of the 1993 flood, simulation of a few selected alternative operations were made using the SAYRED and CORSIM reservoir simulation computer models.

Saylorville Reservoir

In the case of Saylorville Reservoir, an alternative was studied for which a maximum non-growing season release of 16,000 cfs would be implemented year round to determine if storage could have been preserved lessening the impact of severe events like the 1993 flood. Presently 12,000 cfs is the maximum allowable release during the growing season. Results of that simulation showed no reduction in downstream water levels during the 1993 event. Routine releases above 16,000 cfs would impact both urban and agricultural land downstream, especially when high flows are experienced on the Raccoon River. Therefore releases higher than 16,000 cfs were not considered.

Red Rock Reservoir

Red Rock Reservoir has more flood control storage and greater controlled outlet capacity than either Saylorville or Coralville Reservoirs. Red Rock Reservoir reached a peak elevation of 782.7 feet in 1993. The maximum release was 104,500 cfs. Simulation of the 1993 flood with relaxed downstream constraints and a modified large magnitude flood operation schedule, showed that if the release rate of 30,000 cfs when the pool reached elevation 775.0 feet were increased to 70,000 cfs at elevation 775.5 feet, that the pool could have been held below elevation 782.0 feet without any further increase in outflow for this event. Full flood control pool is elevation 780.0 feet. Although simulation of this alternative shows a dramatic reduction in outflow during 1993, a release of 70,000 cfs still causes significant flooding downstream. At releases near 70,000 cfs large expanses of

agricultural land become inundated. Levees protecting downstream communities also are put in jeopardy. Had this plan been in effect for the entire period of record, 1917 through 1993, releases of this magnitude would have been required 4 times. The maximum release prior to 1993 was 40,000 cfs. A plan such as this would negate many of the benefits the reservoir provides during more frequent, less rare, flood events.

Coralville Reservoir

During 1993 Coralville reservoir reached a peak elevation of 716.7 feet with a maximum outflow of 25,800 cfs. A discharge of this magnitude was required since the reservoir had exceeded its designated flood control capacity and was utilizing surcharge storage. As a result some homes, businesses, institutional and critical facilities in the floodplain were impacted downstream in Iowa City. Simulation of two alternatives were conducted to determine if any modifications to the current regulation plan might have lessened the impact of the 1993 flood below Coralville Reservoir. Alternative A, would increase the maximum non-emergency release to 13,000 cfs and increase the release to 16,000 cfs at elevation 695.0 feet which would be held until the lake elevation reached the spillway crest. Presently, non-emergency maximum releases are capped at 6,000 cfs during the growing season and 10,000 cfs the rest of the year. The current emergency release schedule begins at elevation 707.0 feet. Simulation of the 1993 flood following the modified release rules show that the peak pool elevation reached would have been 712.2 feet with a maximum release of 20,000 cfs.

Alternative B, calls for a 10,000 cfs maximum release up to elevation 693.0 feet at which time the release would be increased to 15,000 cfs. As rapidly as possible after reaching lake elevation 693.0 feet, the outflow would be increased from 15,000 cfs to 17,000 cfs and held until water flows over the spillway. Simulation of this alternative operation resulted in a peak pool elevation of 711.6 feet during 1993, 0.4 feet below the spillway crest. The maximum outflow would have been 17,000 cfs.

Although lower reservoir elevations and release rates would have been achieved during the 1993 flood, as in the case of Red Rock Reservoir, higher releases at lower reservoir elevations do not optimize available storage. This negates benefits accrued during more frequent, less severe flood events. Prior to 1993, the maximum release from Coralville Reservoir was 13,000 cfs. Simulation of a 90 year record, 1904 through 1993 under both alternative operation plans showed that releases greater than 13,000 cfs would have been required numerous times. Table RI-10 shows how frequently higher release rates would have been required under each alternative in comparison with the current plan for the simulated 90 year period of record.

Table RI-10

Simulation of Alternative Regulation Plans for Coralville Reservoir
Discharge vs.Frequency

Release Rate	Current Plan	Alternative A	Alternative B
13,000 cfs	2	31	15
14,000 cfs	1	11	14
15,000 cfs	1	10	11
16,000 cfs	1	8	4
17,000 cfs	1	1	1
20,000 cfs	1	1	0

Outflows above 13,000 cfs require closure of several roads and inundate some residences downstream of the reservoir. Routine release of flows in the range cited above may require relocation or floodproofing of certain structures in the floodplain. More frequent flooding would likely limit agricultural production. Presently, the Corps of Engineers does not have any real estate interest in lands downstream of the reservoir. Therefore, discharges above 6,000 cfs would require an authority to purchase real estate. Any change to the regulation plan would have to address the tradeoff in benefits which would be required to place greater emphasis on reduction of damages during extreme, rare, flood events, in lieu of a broad range of flood events. At present, funding is being sought to perform a comprehensive study of Coralville Reservoir to evaluate the tradeoff between urban and rural flood control benefits and address changed physical, economic, and hydrologic conditions which have occurred since the reservoir began operation.

In general, changes to reservoir operation at Corps reservoirs during the 1993 flood would have had little effect in lessening the impacts of the flood, due to the severity and duration of the event. Although some of the hypothetical alternatives studied showed reductions in peak reservoir stages and outflows, no authority exists to implement such changes without a comprehensive investigation of basin wide impacts. Reservoir operating plans are designed to optimize benefits for the projects intended and authorized purpose over a wide range of conditions. These optimizations are based on either historic or synthetic flood simulations or some downstream control point to determine the optimum operating conditions which maximize benefits. Reservoir releases during floods are made based on known hydrologic conditions or forecasts. In the case of the 1993 flood, downstream hydrologic conditions were a moving target for forecasters. In most cases, revised operating plans would have only been of benefit had there been foresight of the ensuing conditions that sometimes changed after decisions concerning releases were made days or even weeks or months earlier. An optimal reservoir operation plan should be based on an entire range of flood events, not a single extreme event.

ADDITIONAL RESERVOIRS

During the 1960's and 1970's, construction of reservoirs in the Raccoon and Skunk River basins were proposed. Analyses for this assessment were conducted to determine what, if any impact these reservoirs would have had on the 1993 flood had they been constructed.

Jefferson Reservoir

In 1966, an economically justified plan was formulated by the Corps of Engineers to construct the Jefferson Reservoir. The plan is described in the report *Des Moines River, Interim Review of Reports for Flood Control and Other Purposes,* Jefferson Reservoir, U.S. Army Engineer District, Rock Island, 28 January 1966.

The dam site was located in Greene County about 10 miles upstream from Jefferson, Iowa on the North Raccoon River. The drainage area above the dam site is 1,552 square miles. The reservoir would be 24 miles in length covering portions of Greene, Carroll and Calhoun Counties. The narrow valley floodplain has a maximum width of 1/2 mile. At elevation 1090 feet, which is full capacity, the area that would be inundated is approximately 10,700 acres. Total reservoir capacity would be 312,000 acre-feet which includes storage allocated for sediment, water quality, and flood control. Of this, 130,700 acre-feet which is equivalent to 1.6 inches of runoff over the basin, would be allocated for flood control storage. The reservoir would require a total of 15,200 acres of land for the project.

Using the elevation storage relationship and regulation plan outlined in the interim report, 1993 mean daily discharges for the Raccoon River at Jefferson, Iowa were routed through the reservoir to determine if any reduction in peak stages downstream could have been accomplished during the 1993 flood. That analysis revealed that with an optimal operation, the maximum possible reduction in discharge would have been 10,000 cfs. This equates to a one foot reduction in stage in the cities of Des Moines and West Des Moines. However, a reduction of even one foot in stage is probably optimistic. Limited storage capacity allocated for flood control along with the fact that the majority of runoff was contributed by the Middle and South Raccoon Rivers, presented little opportunity for flow reduction on the mainstem of the Raccoon River. Table RI-11 shows monthly runoff volume in terms of multiples of allocated flood control storage capacity.

Table RI-11

Monthly Runoff vs. Multiples of Allocated Flood Control Storage Capacity
Jefferson Reservoir, Raccoon River

Month	Monthly runoff in inches above Jefferson Reservoir	Multiples of Flood Control Capacity
March	1.86	1.2
April	3.62	2.3
May	2.52	1.6
June	2.40	1.5
July	5.40	3.4
August	2.14	1.3
September	1.12	0.7
Total	**19.06**	**12.0**

Table RI-12 shows peak discharges on the North, Middle, South, and mainstem Raccoon Rivers.

Table RI-12

1993 Peak Discharges
at Gaging Stations in the Raccoon River Basin

Location	Peak Discharge in cfs
North Raccoon River nr. Jefferson, Iowa	**16,900**
Middle Raccoon River nr. Panora, Iowa	22,400 *
South Raccoon River at Redfield, Iowa	44,000 *
Raccoon River at Van Meter, Iowa	70,100 *

* Denotes Record Discharge

Since levee projects along the Raccoon River in West Des Moines and Des Moines have been modified to contain the 1993 flood, construction of the Jefferson Reservoir may no longer be economically justified.

Skunk River and Squaw Creek Reservoirs

Congress authorized the Ames Reservoir in the Skunk River basin in 1965. In October 1973, the State of Iowa withdrew support for the project. In 1984, the project was reactivated and studied in a *General Reevaluation Report, Upper Skunk River Basin, Iowa* (Ames Lake), U.S. Army Corps of Engineers, Rock Island District, July 1987.

The General Reevaluation Report examined a variety of reservoir options. On the Skunk River immediately above Ames, Iowa, a number of various sized multi-purpose reservoirs were considered. A reservoir having 51,000 acre-feet of storage allocated to flood control storage, which translates to 3.0 inches of basin runoff, had the greatest benefit to cost ratio.

The Squaw Creek Reservoir site is about 8.6 miles upstream from the confluence with the Skunk River. It was studied as a single-purpose flood control detention dam with a dry reservoir capable of storing 2.4 inches of runoff. The site has a drainage area of 160 square miles. The proposed reservoir would have a length of 4.75 miles and a capacity of 20,500 acre-feet at the spillway crest. Squaw Creek, is subject to flash flooding. The 1993 flood, was the flood of record on Squaw Creek.

Explicit regulation plans for Ames and Squaw Creek Reservoirs were unavailable which prohibited routing of the 1993 flood through either reservoir. However, examination of runoff volumes vs. allocated flood control storage, indicates these reservoirs would have probably had minimal impacts on 1993 flood peaks. Table RI-13 lists monthly runoff versus multiples of allocated flood control storage capacity for both the Ames Lake 3.0-inch project and Squaw Creek reservoir.

Table RI-13

Flood Plain Management Assessment
Monthly Runoff vs. Allocated Flood Control Storage Capacity
Ames Lake and Squaw Creek

Month	Monthly runoff in inches above Ames Reservoir	Multiples of Flood Control Capacity	Monthly runoff in inches above Squaw Creek Reservoir	Multiples of Flood Control Capacity
March	2.94	1.0	3.10	1.3
April	2.51	0.8	2.11	0.9
May	2.24	0.7	2.55	1.1
June	4.52	1.5	4.50	1.9
July	9.62	3.2	12.03	5.0
August	6.52	2.2	6.66	2.8
September	2.01	0.7	2.11	0.9
Total	**30.36**	**10.1**	**33.06**	**13.9**

As shown above, during the month of July alone, runoff exceeded storage capacity at both sites by 3 to 5 times respectively. In summary, the volume of the 1993 flood far exceeded the capacity set aside for flood control storage at these sites although some reduction in flood peaks may have been possible depending on the regulation strategy.

RUNOFF REDUCTION MEASURES

UNET simulations assuming runoff volume reductions of 5% and 10 % were made to see how Mississippi River water surface profiles of the 1993 flood would have been affected. The purpose of the analysis was to evaluate the sensitivity of Mississippi River stages to reductions in runoff, and in no way implies that 5% to 10% reduction in runoff volume is achievable. Depending on individual drainage basin characteristics, some tributary basins could store more than 10 percent of the basin runoff volume while some have little or no upland retention storage available. To simplify UNET modeling, all the inflow hydrographs were reduced by an equal amount. In reality, runoff reduction would not be distributed equally over the total inflow hydrograph. In most cases increased runoff retention would have a major impact on the shape of the inflow hydrograph at the beginning of the 1993 event and would have little or no impact on peak discharges and stages. Table RI-10 compares the 1993 peak flood stages with computed stages coincident with the assumed volume reductions. . Levee district overtopping information is shown back on table RI-7. Stage and flow hydrographs are shown on plates RI-34 through RI-41

SUMMARY

The hydraulic analysis conducted by the Rock Island District of all the alternatives discussed above is intended to indicate trends of what might be expected to occur if those alternatives were implemented. Stage differences between the 1993 flood and the various action alternatives are computed to the nearest one-tenth of a foot. However, because of the assumptions and data limitations inherent in the model, stage differences at some locations may vary by as much as one foot.

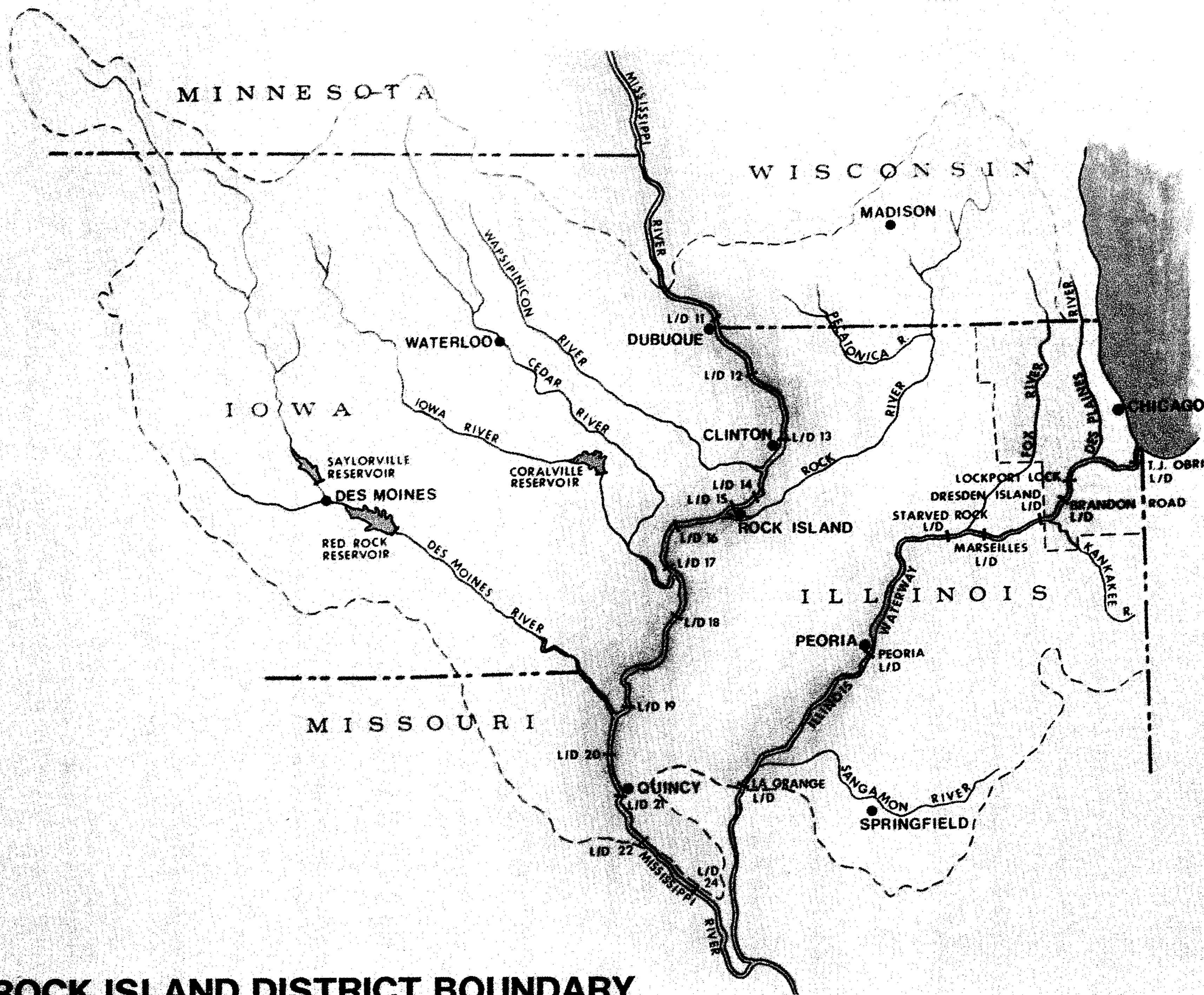
REFERENCES

Des Moines River, Interim Review of Reports for Flood Control and Other Purposes, Jefferson Reservoir, U.S. Army Engineer District, Rock Island, 28 January 1966.

General Reevaluation Report, Upper Skunk River Basin, Iowa, Ames Lake, U.S. Army Corps of Engineers, Rock Island District, July 1987.

Post-Flood Report, The Great Flood of 1993, Upper Mississippi River Basin, Appendix B, U.S. Army Corps of Engineers, Rock Island District, September, 1994.

UNET, One-Dimensional Unsteady Flow Through a Full Network of Open Channels, Hydrologic Engineering Center, Davis California, September 1992.



ROCK ISLAND DISTRICT BOUNDARY

NOV 1992 - APR 1993 PRECIPITATION

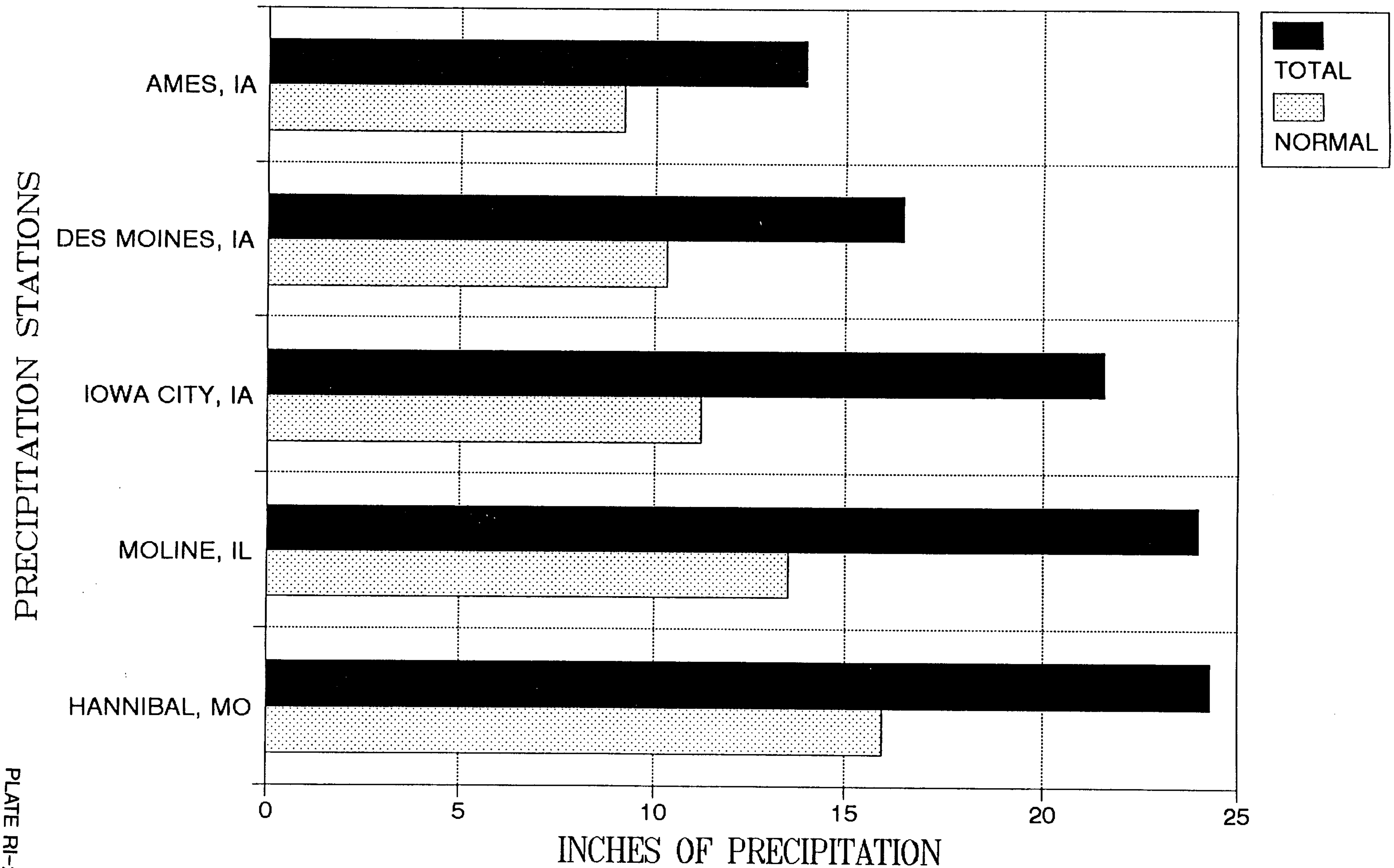


PLATE RI-2

19950725 032

The missing page in this report (Plate R1-3) was misnumbered.
The original copy that went to the printer was checked and this was discovered, per Jean Schmidt of USACE, St. Paul, MN.

UPPER MISSISSIPPI RIVER SYSTEM

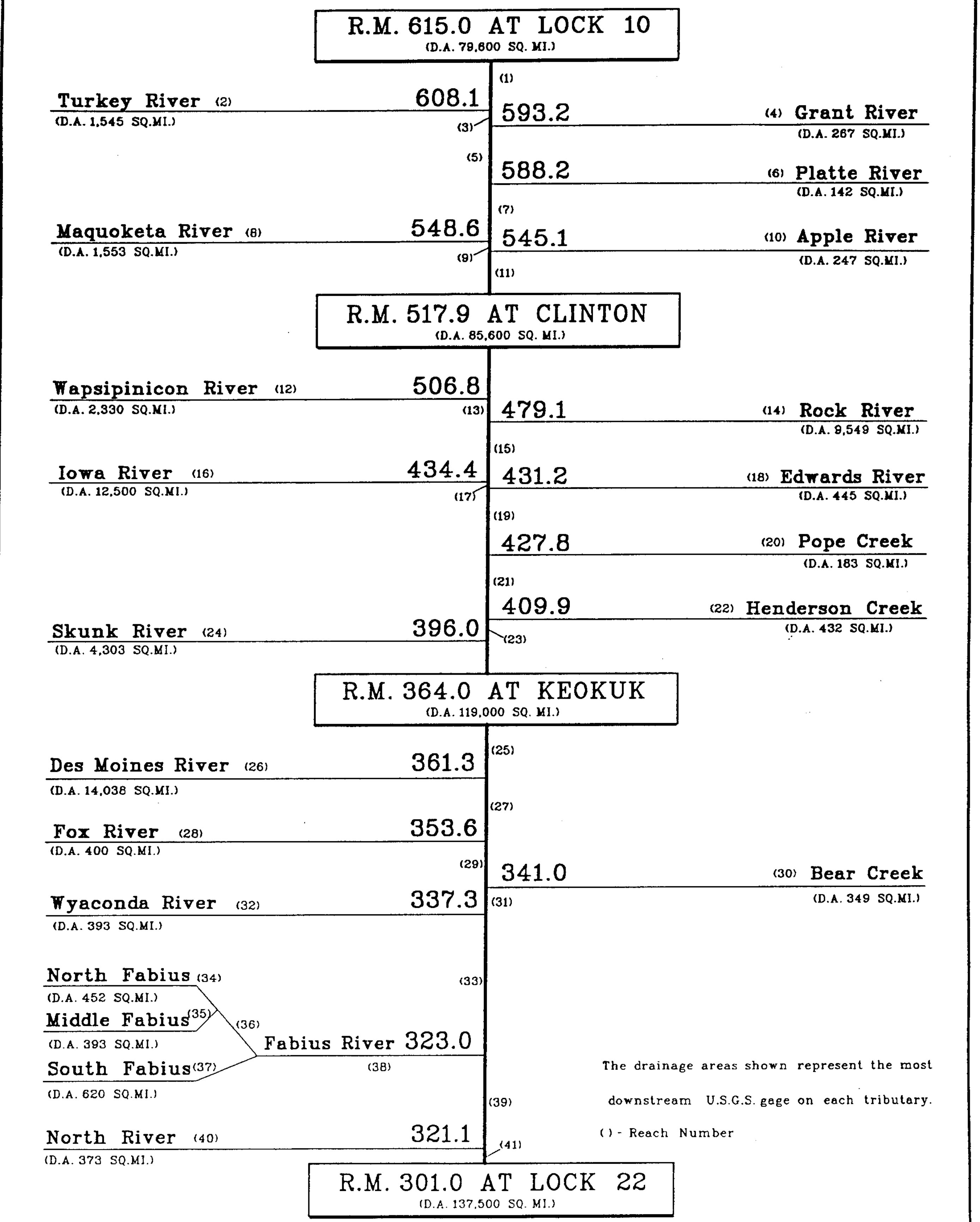
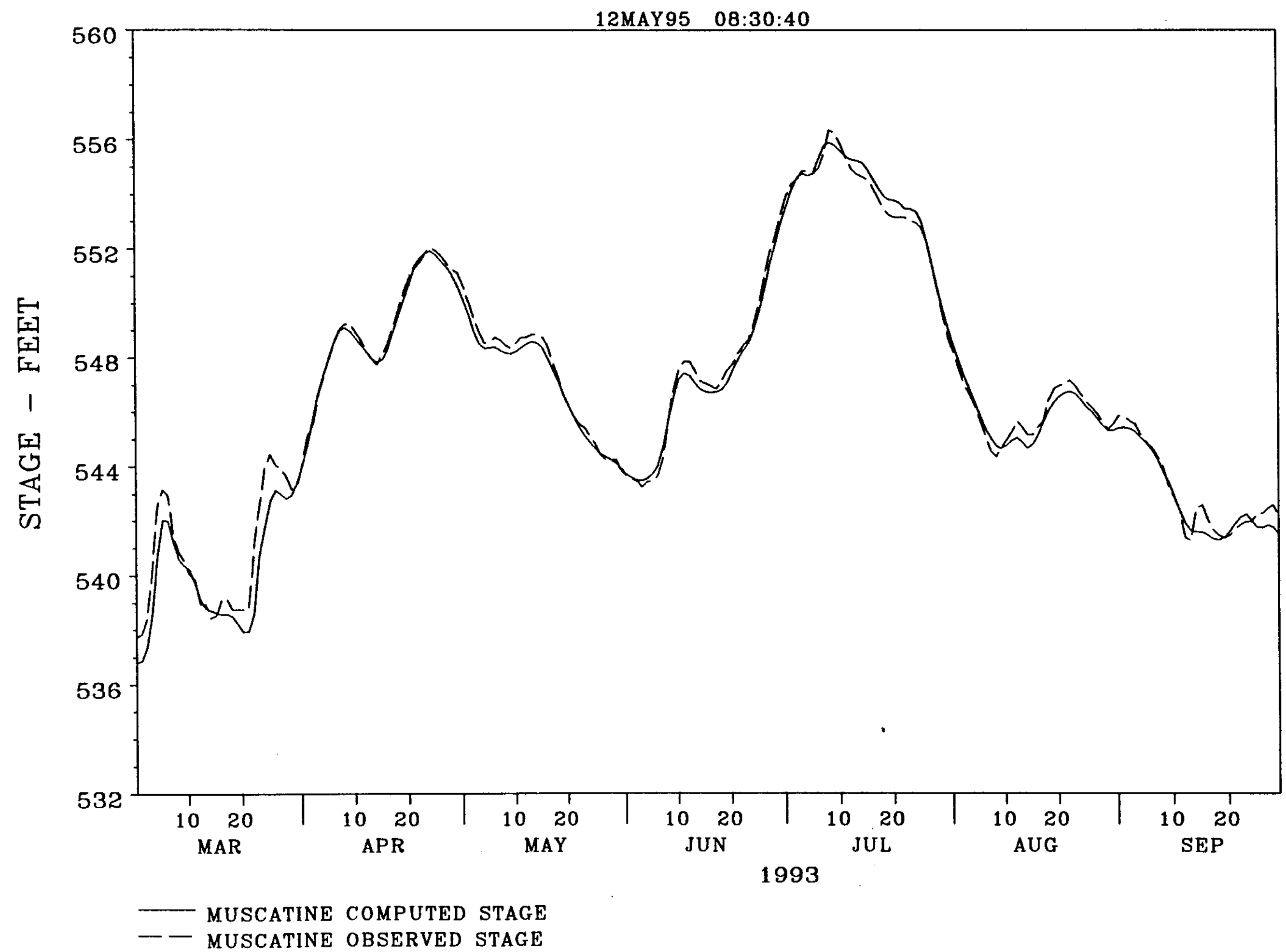
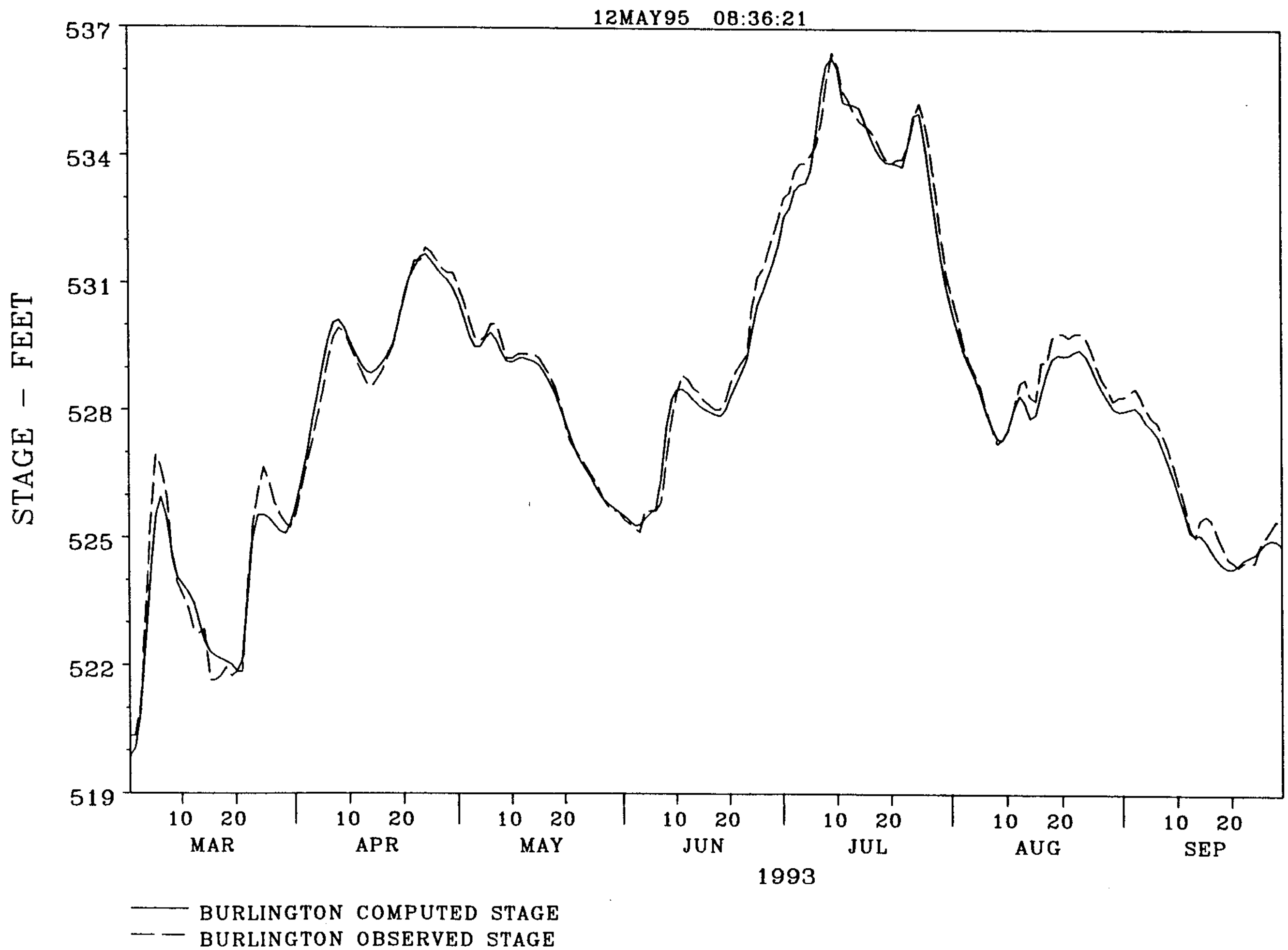


PLATE RI-4

MISSISSIPPI RIVER
MUSCATINE, IOWA – R.M. 455.2
COMPUTED VS. OBSERVED



MISSISSIPPI RIVER
BURLINGTON, IOWA – R.M. 403.1
COMPUTED VS. OBSERVED



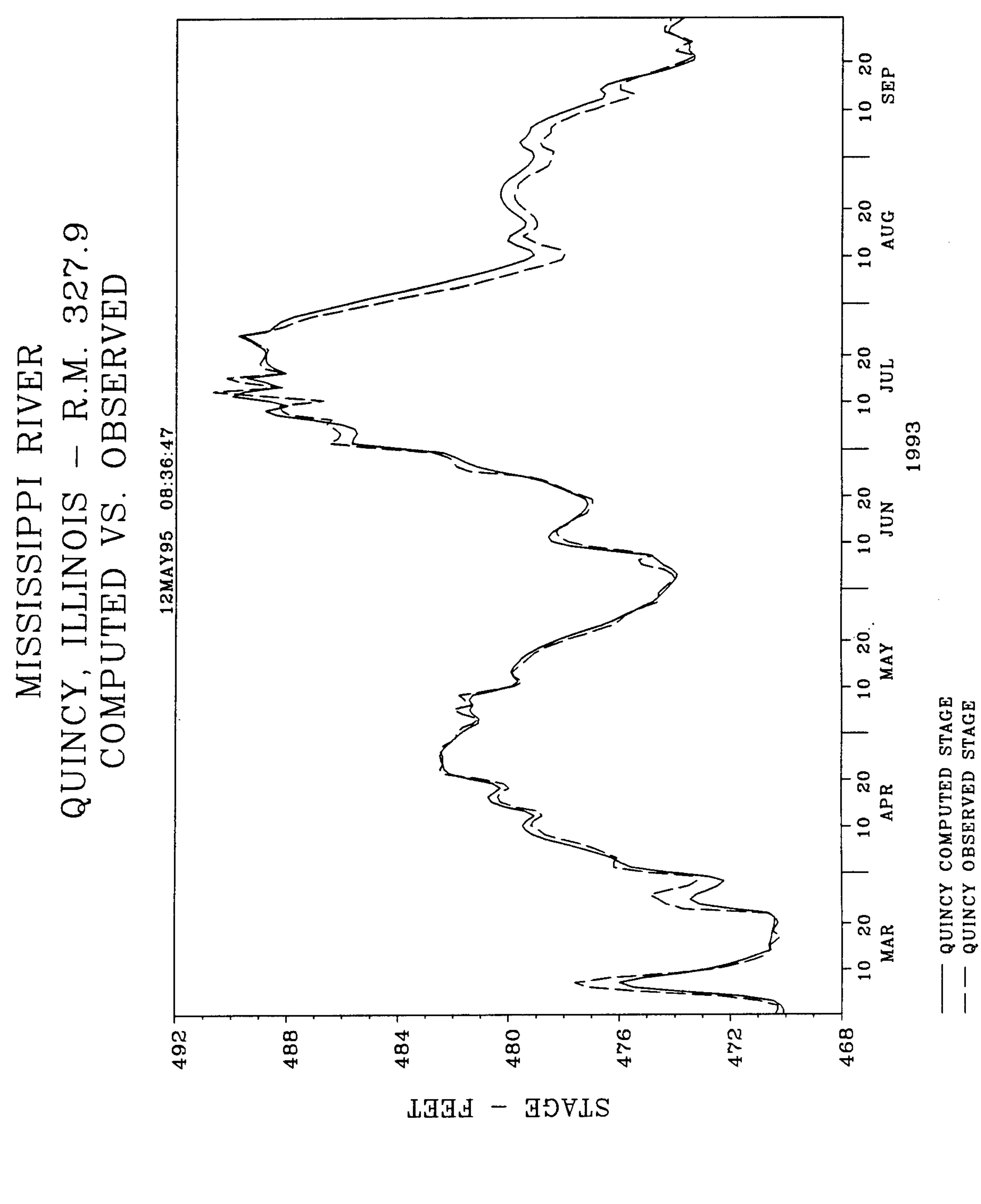
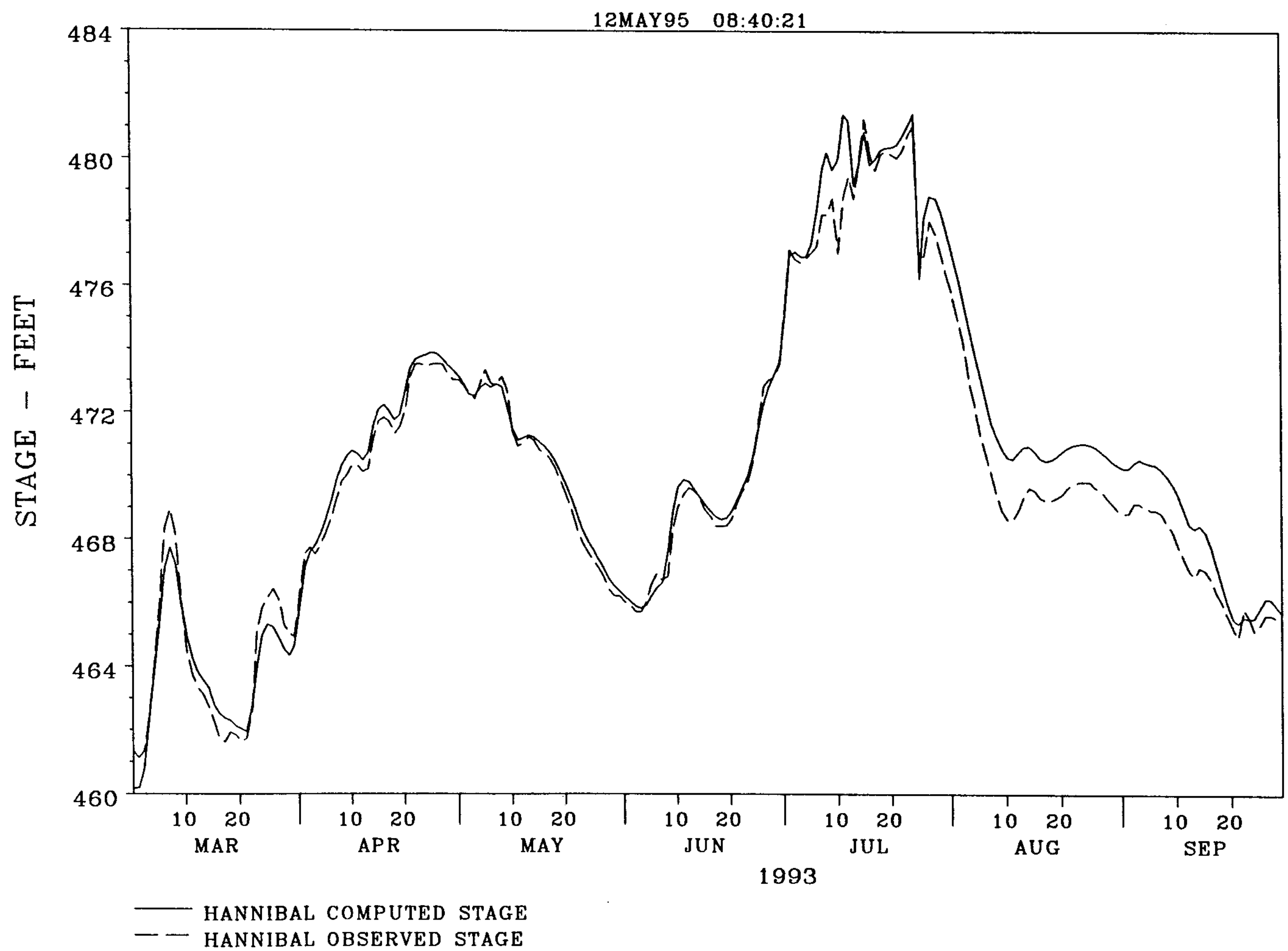
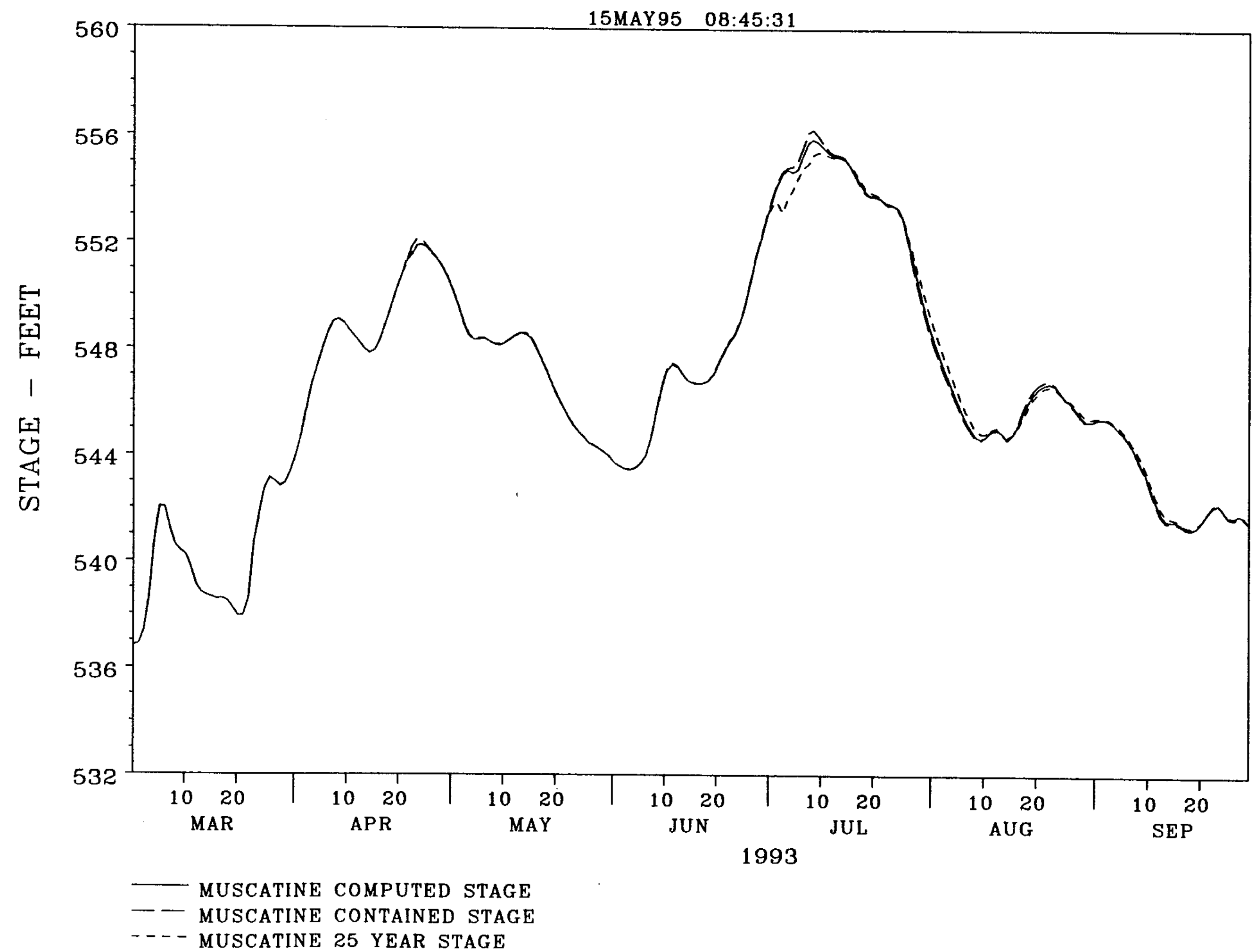

MISSISSIPPI RIVER
QUINCY, ILLINOIS - R.M. 327.9
COMPUTED VS. OBSERVED
12MAY95 08:36:47
STAGE - FEET
492
488
484
480
476
472
468
10 20
MAR
10 20
APR
10 20
MAY
10 20
JUN
10 20
JUL
10 20
AUG
10 20
SEP
1993
QUINCY COMPUTED STAGE
QUINCY OBSERVED STAGE

PLATE RI-7

MISSISSIPPI RIVER
HANNIBAL, MISSOURI – R.M. 309.9
COMPUTED VS. OBSERVED



MISSISSIPPI RIVER
MUSCATINE, IOWA – R.M. 455.2
25 YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



MISSISSIPPI RIVER
BURLINGTON, IOWA - R.M. 403.1
25 YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES

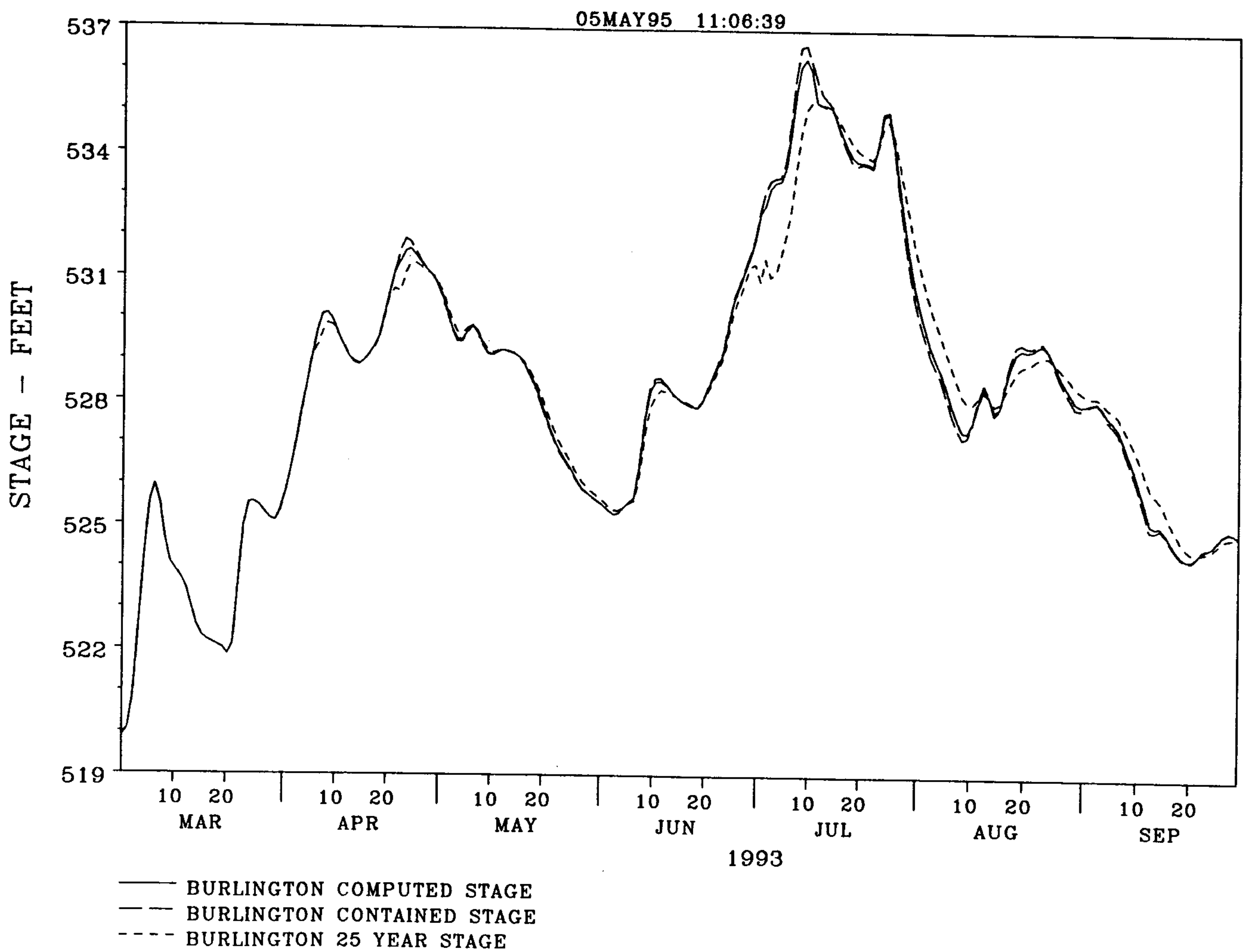
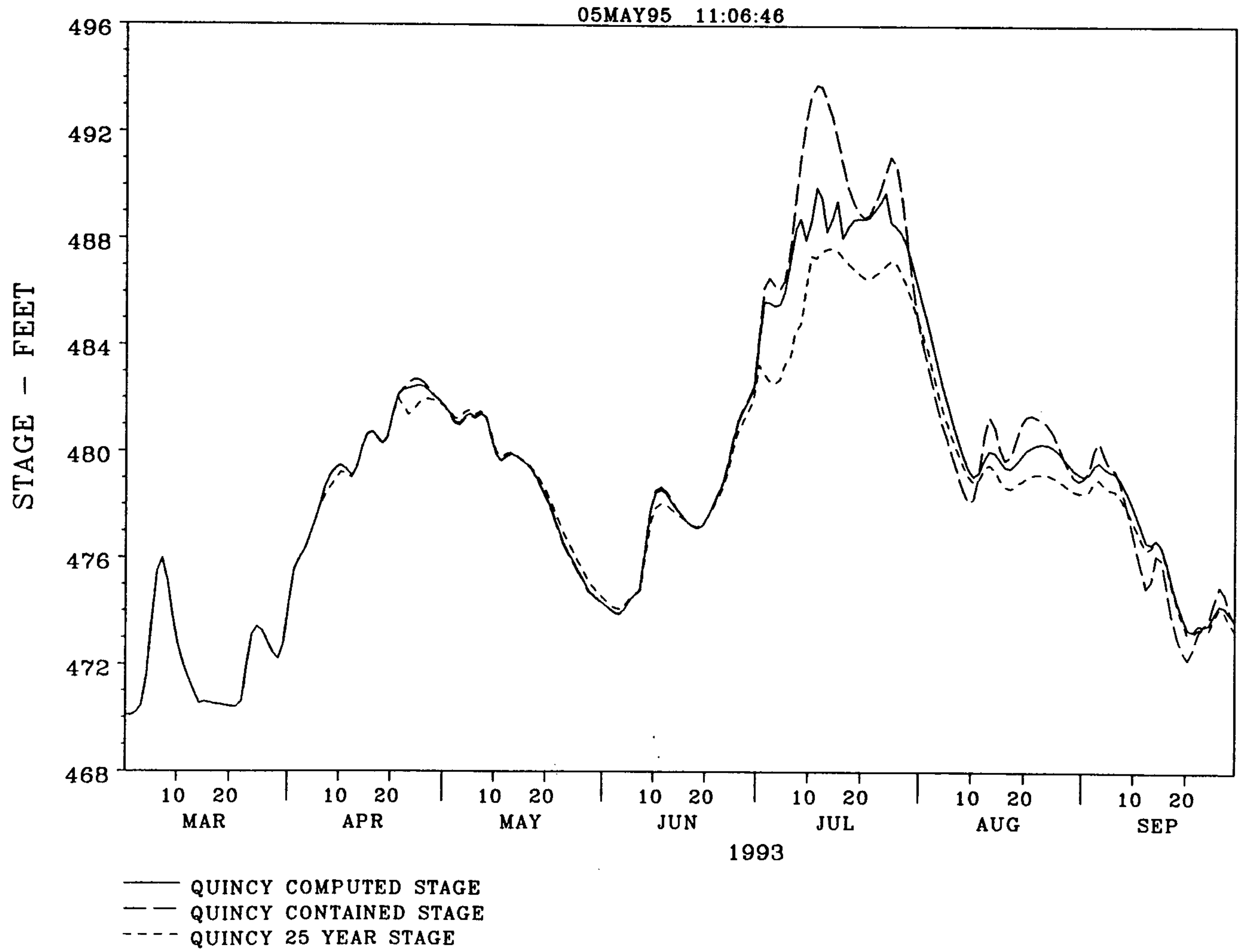


PLATE RI-10

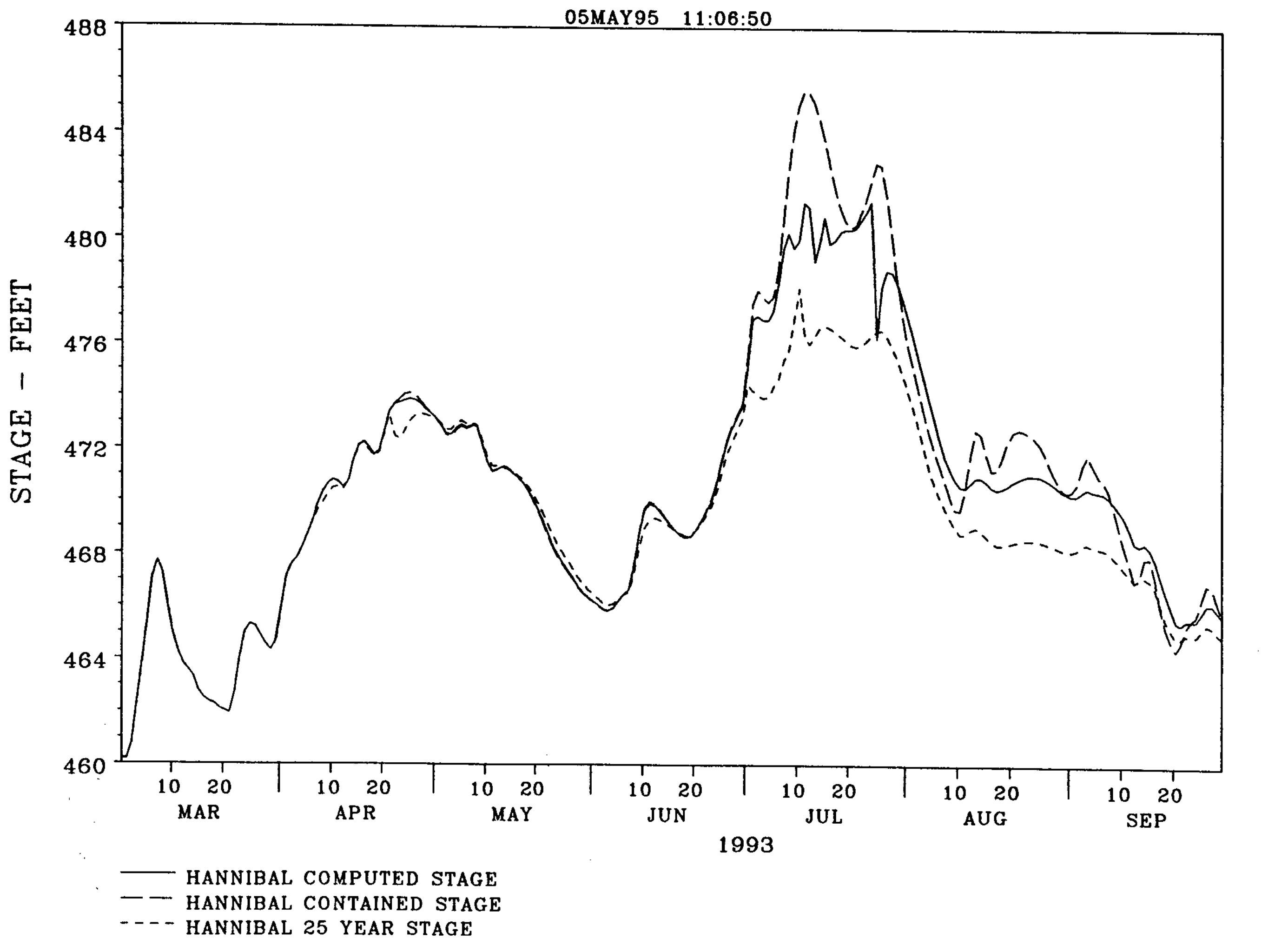
MISSISSIPPI RIVER
QUINCY, ILLINOIS – R.M. 327.9
25 YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



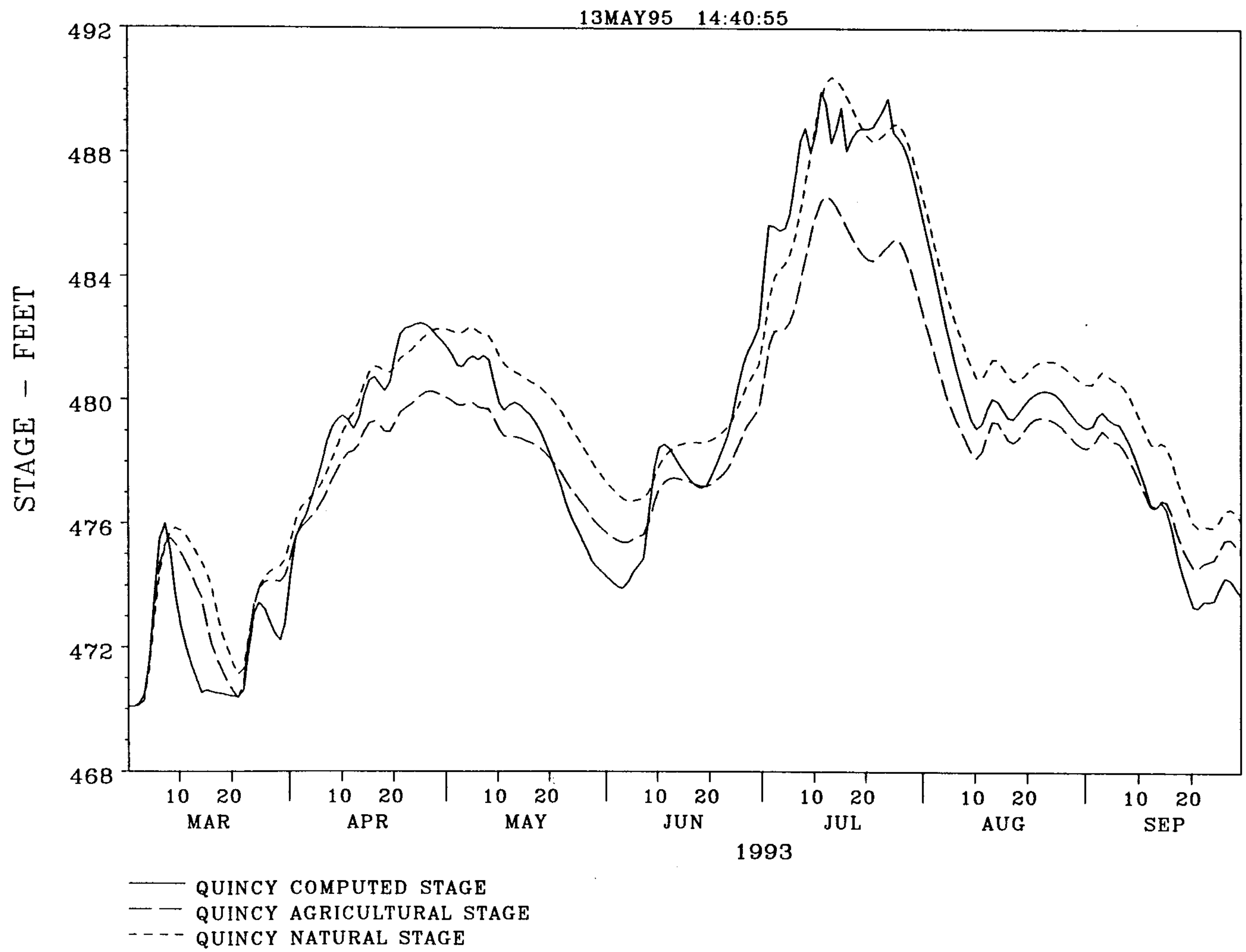
MISSISSIPPI RIVER

HANNIBAL, MISSOURI – R.M. 309.9

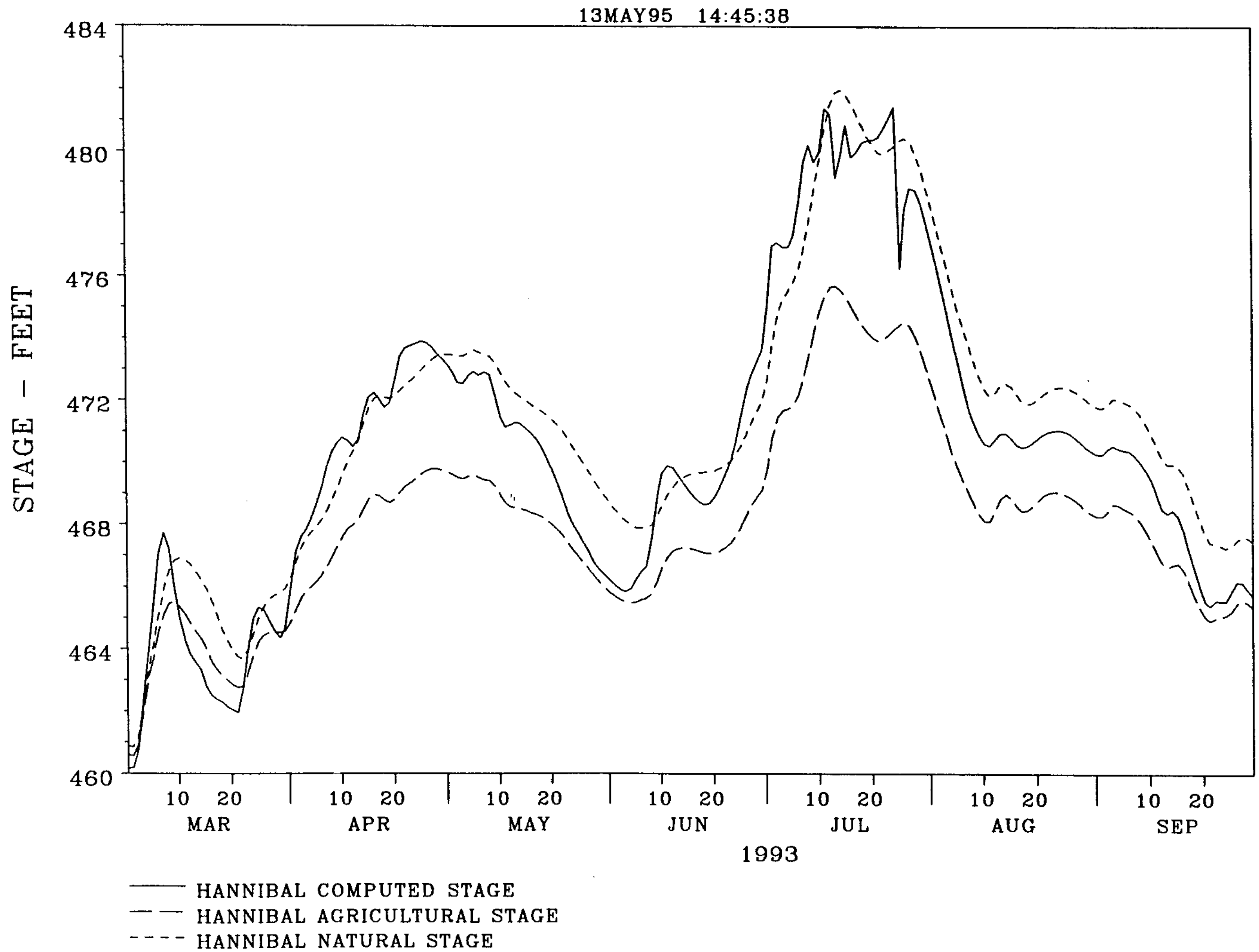
25 YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



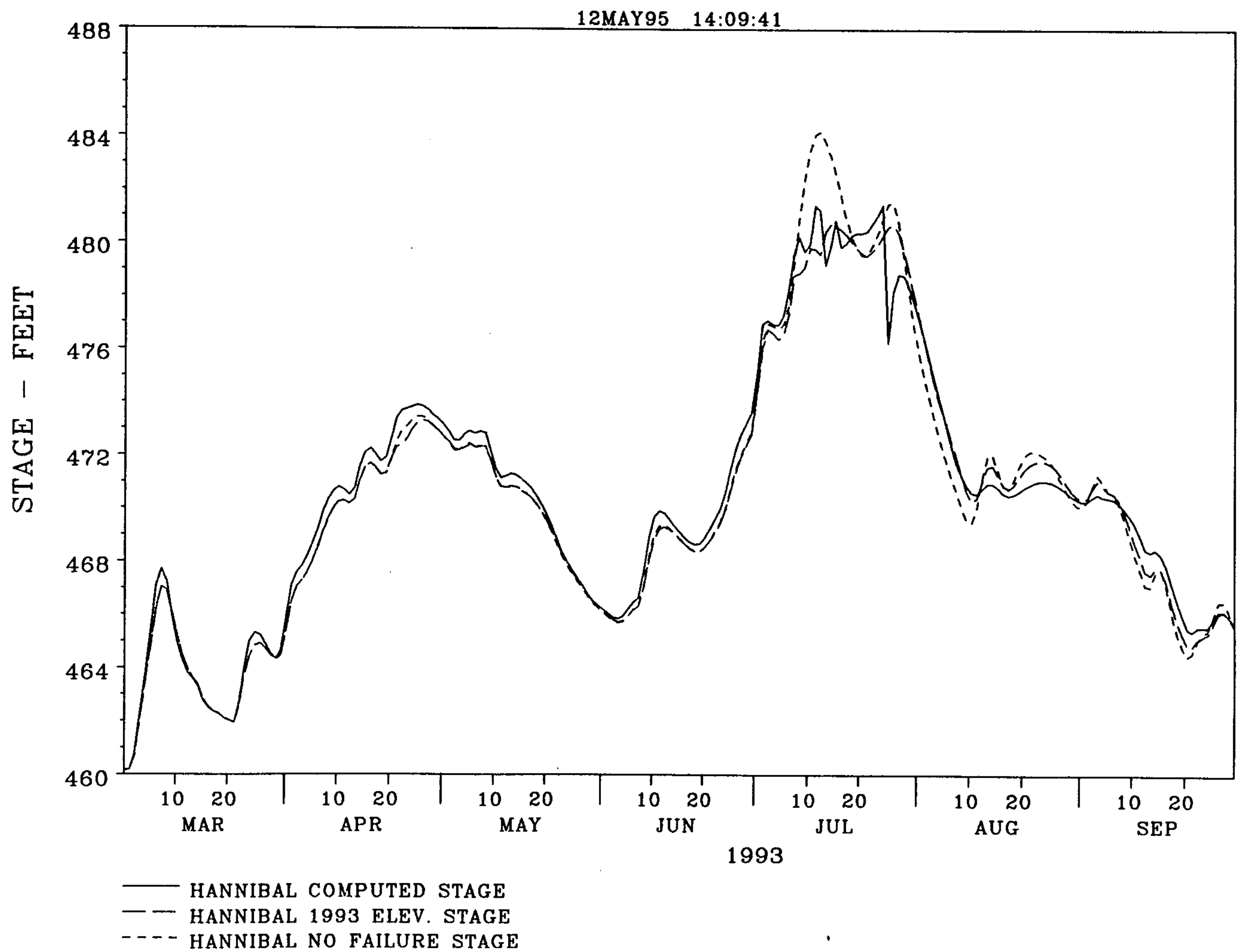
MISSISSIPPI RIVER
QUINCY, ILLINOIS – R.M. 327.9
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

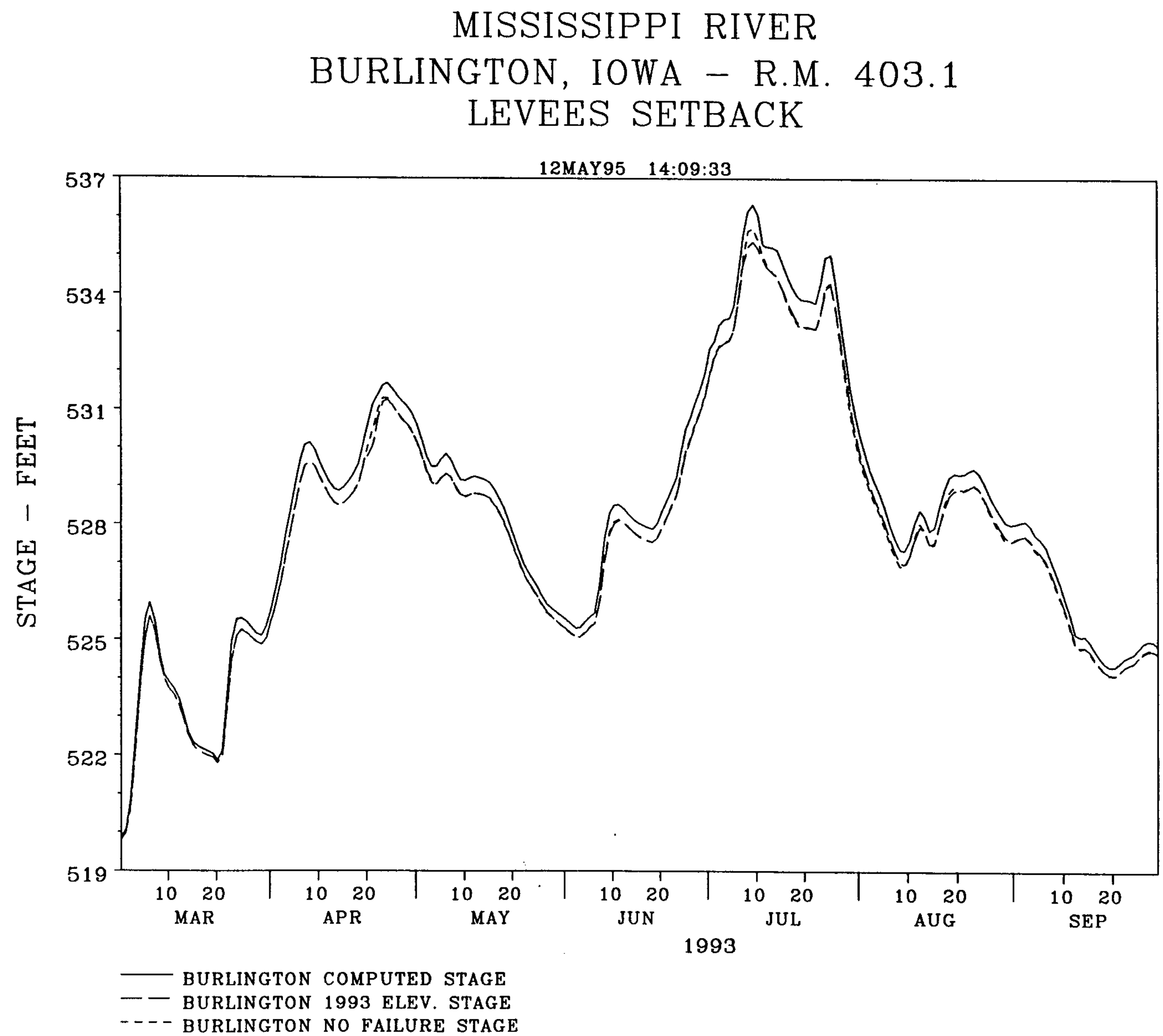


MISSISSIPPI RIVER
HANNIBAL, MISSOURI – R.M. 309.9
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS



MISSISSIPPI RIVER
HANNIBAL, MISSOURI – R.M. 309.9
LEVEES SETBACK





MISSISSIPPI RIVER
BURLINGTON, IOWA - R.M. 403.1
LEVEES SETBACK
12MAY95 14:09:33
STAGE - FEET
537
534
531
528
525
522
519
10 20 MAR
10 20 APR
10 20 MAY
10 20 JUN
10 20 JUL
10 20 AUG
10 20 SEP
1993
BURLINGTON COMPUTED STAGE
BURLINGTON 1993 ELEV. STAGE
BURLINGTON NO FAILURE STAGE

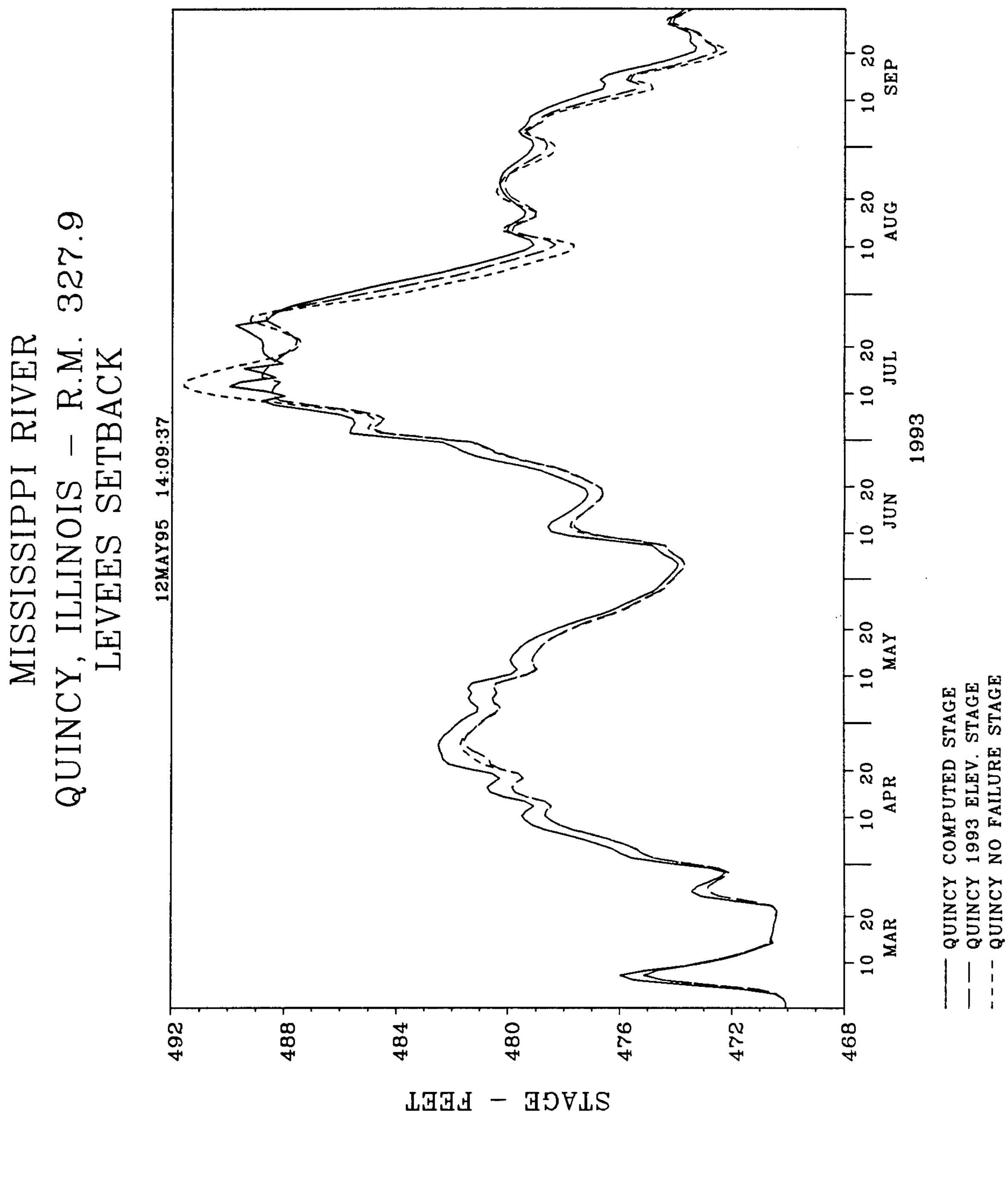
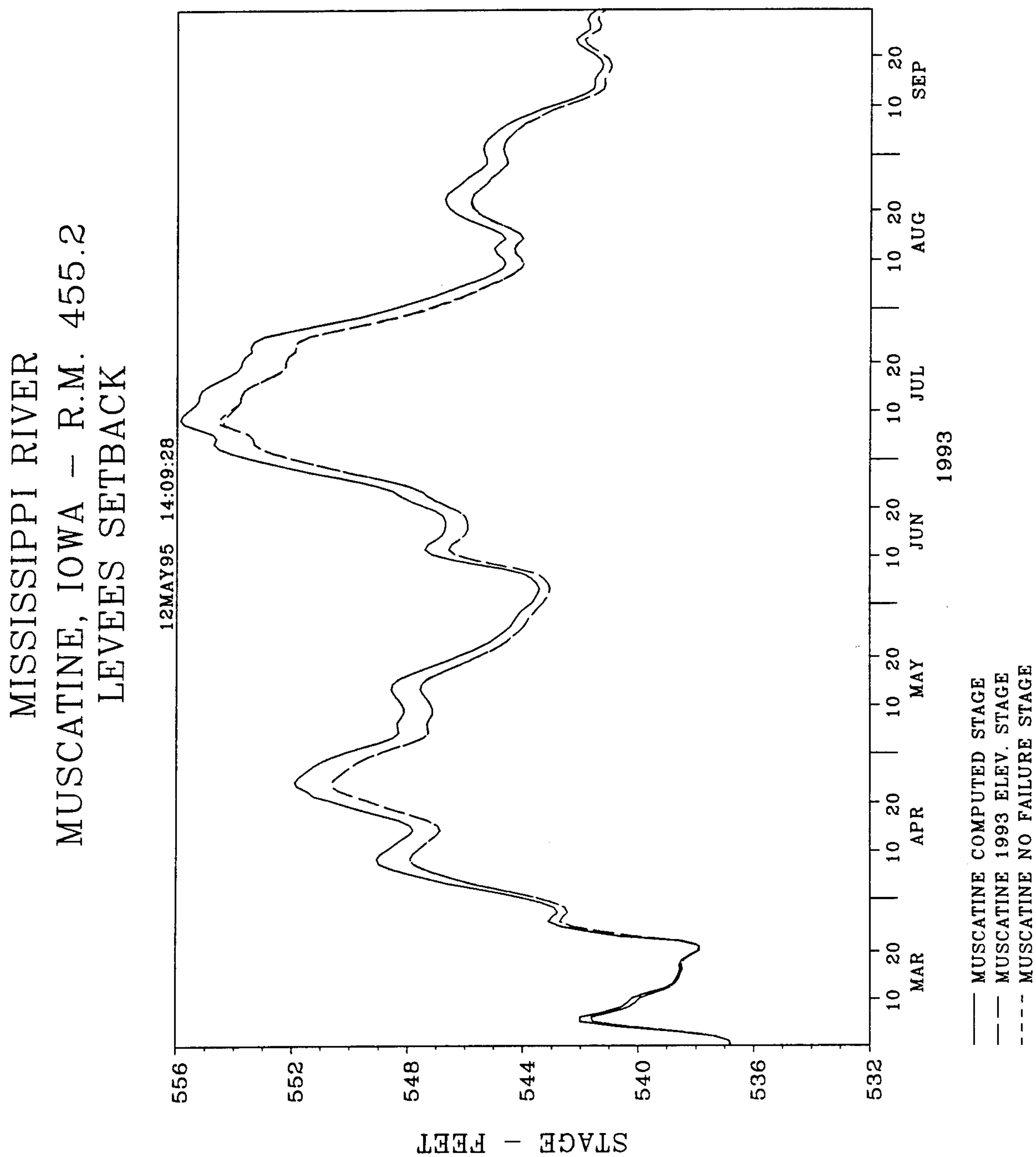

MISSISSIPPI RIVER
QUINCY, ILLINOIS - R.M. 327.9
LEVEES SETBACK
12MAY95 14:09:37
STAGE - FEET
492
488
484
480
476
472
468
10 20
MAR
10 20
APR
10 20
MAY
10 20
JUN
10 20
JUL
10 20
AUG
10 20
SEP
1993
QUINCY COMPUTED STAGE
QUINCY 1993 ELEV. STAGE
QUINCY NO FAILURE STAGE

PLATE RI-19


MISSISSIPPI RIVER
MUSCATINE, IOWA — R.M. 455.2
LEVEES SETBACK
12MAY95 14:09:28
STAGE - FEET
556
552
548
544
540
536
532
10 20 MAR
10 20 APR
10 20 MAY
10 20 JUN
10 20 JUL
10 20 AUG
10 20 SEP
1993
MUSCATINE COMPUTED STAGE
MUSCATINE 1993 ELEV. STAGE
MUSCATINE NO FAILURE STAGE

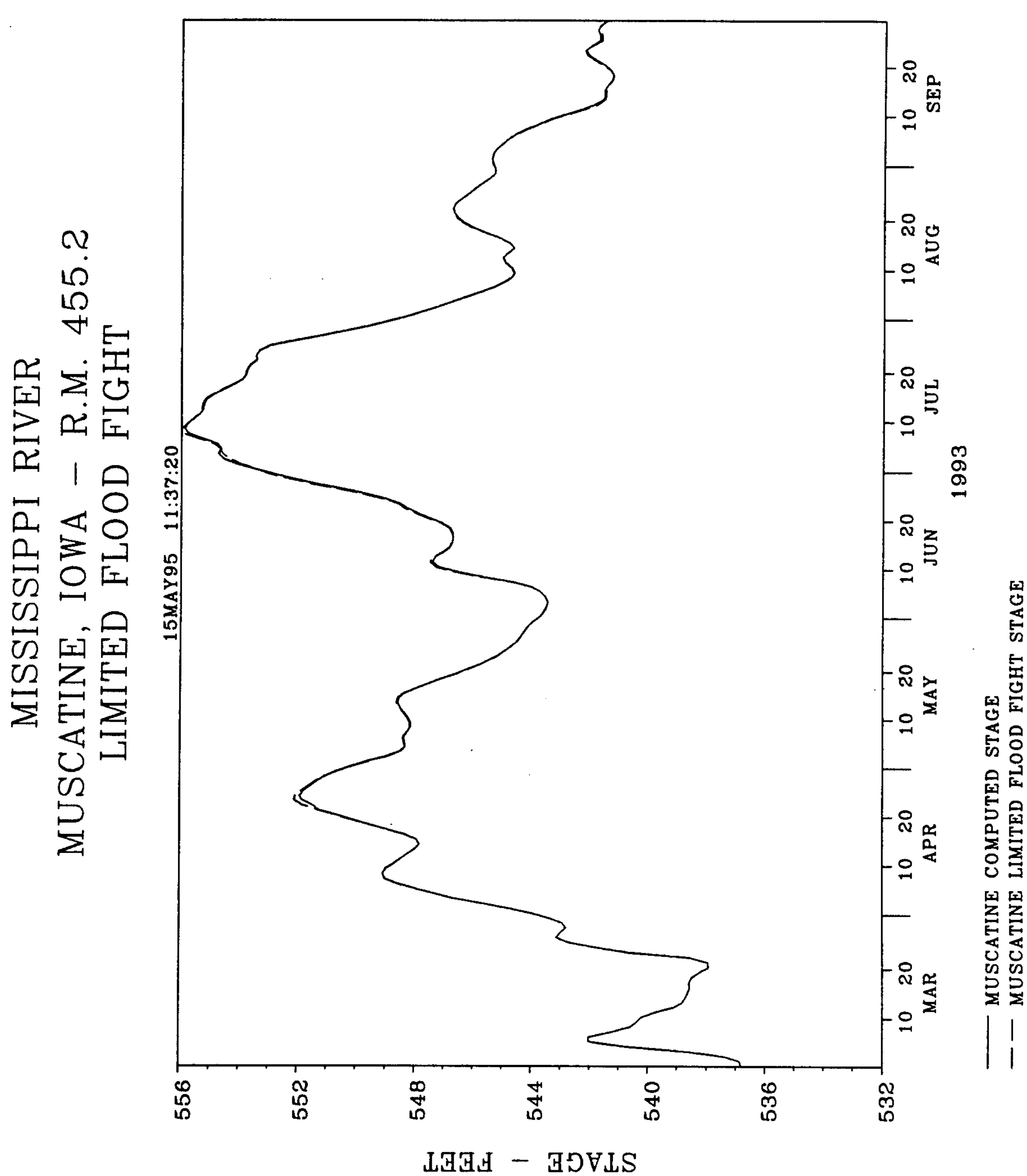
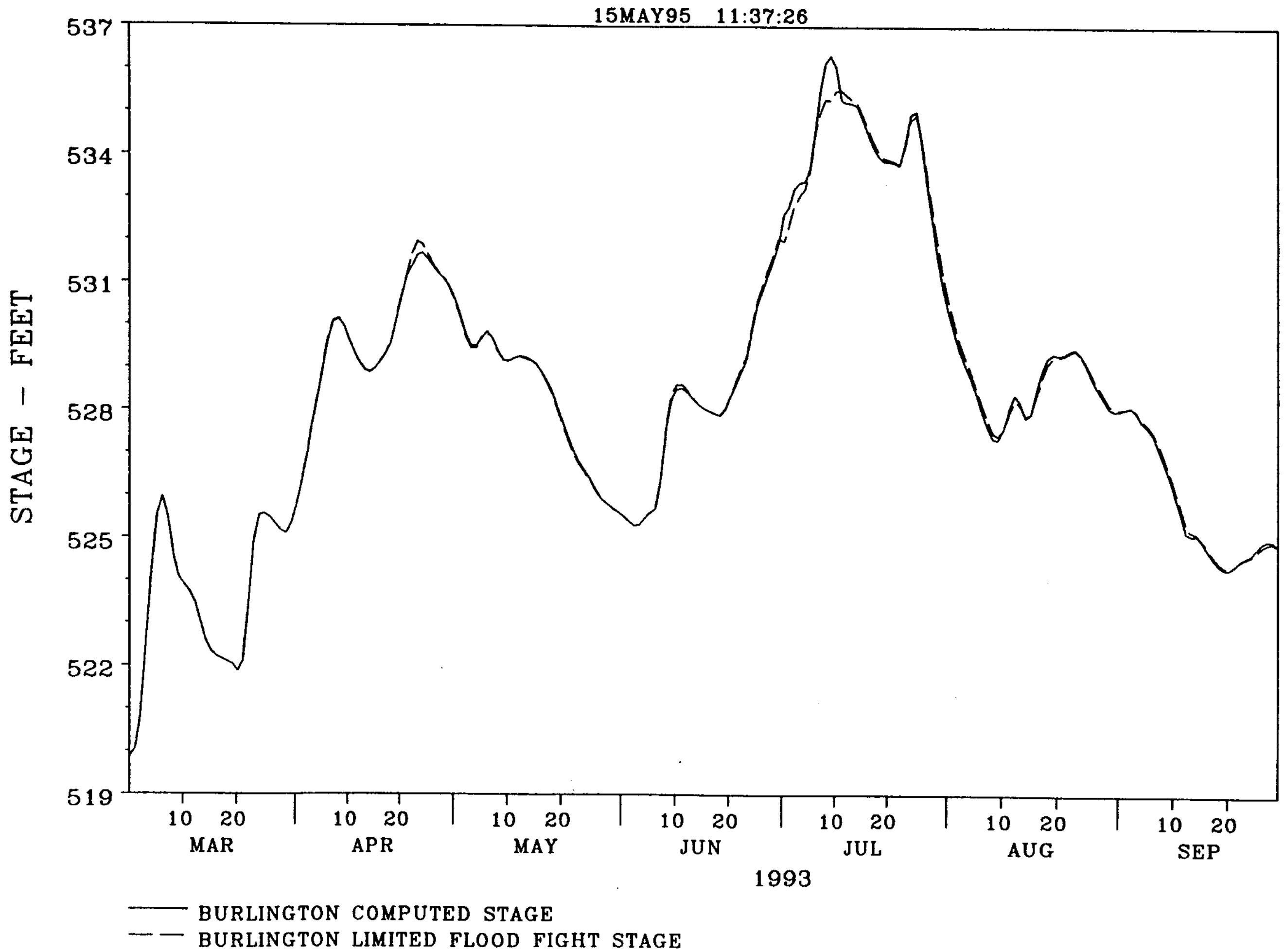
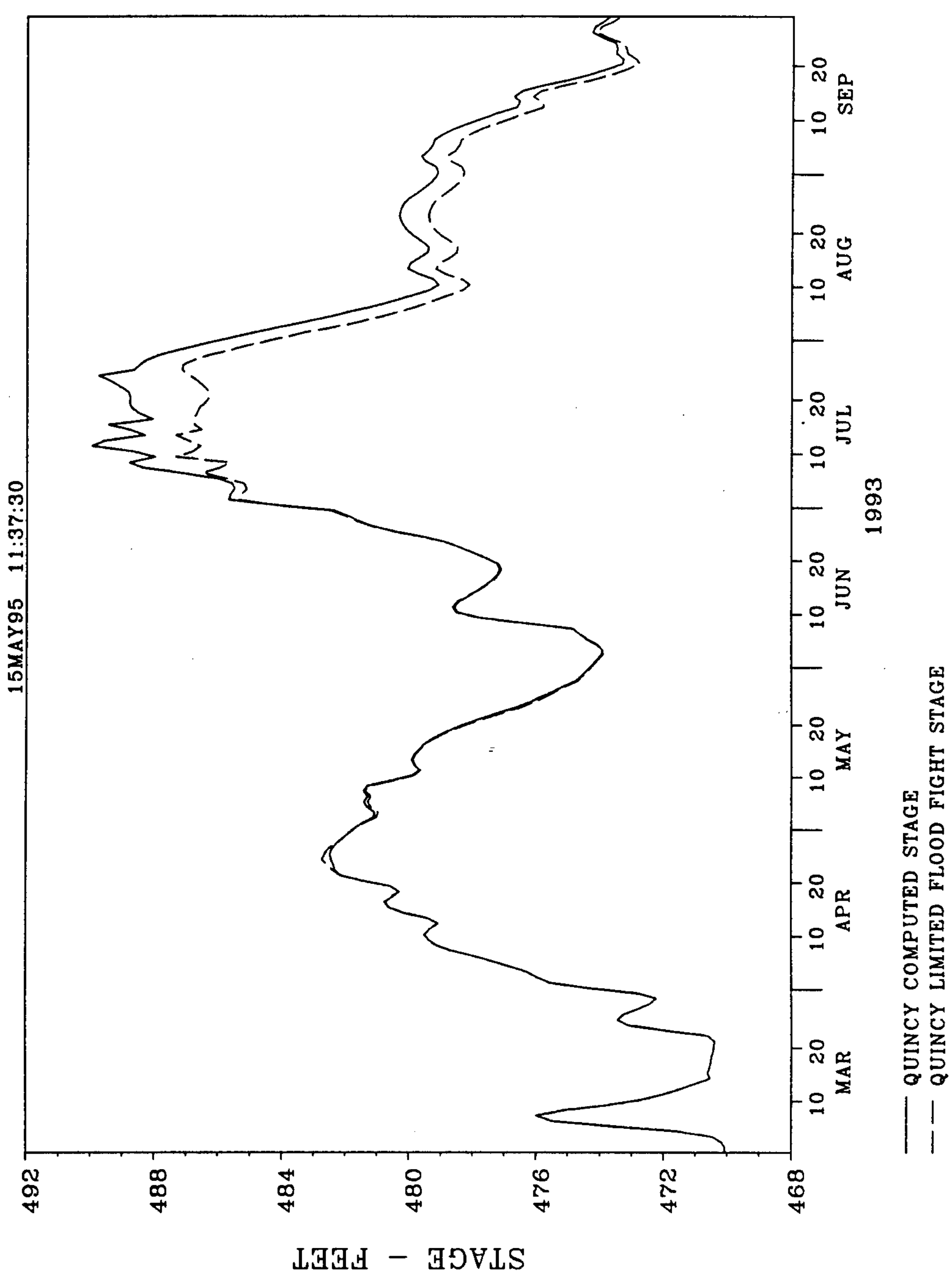

MISSISSIPPI RIVER
MUSCATINE, IOWA - R.M. 455.2
LIMITED FLOOD FIGHT
15MAY95 11:37:20
STAGE - FEET
556
552
548
544
540
536
532
10 20
MAR
10 20
APR
10 20
MAY
10 20
JUN
10 20
JUL
10 20
AUG
10 20
SEP
1993
MUSCATINE COMPUTED STAGE
MUSCATINE LIMITED FLOOD FIGHT STAGE

PLATE RI-21

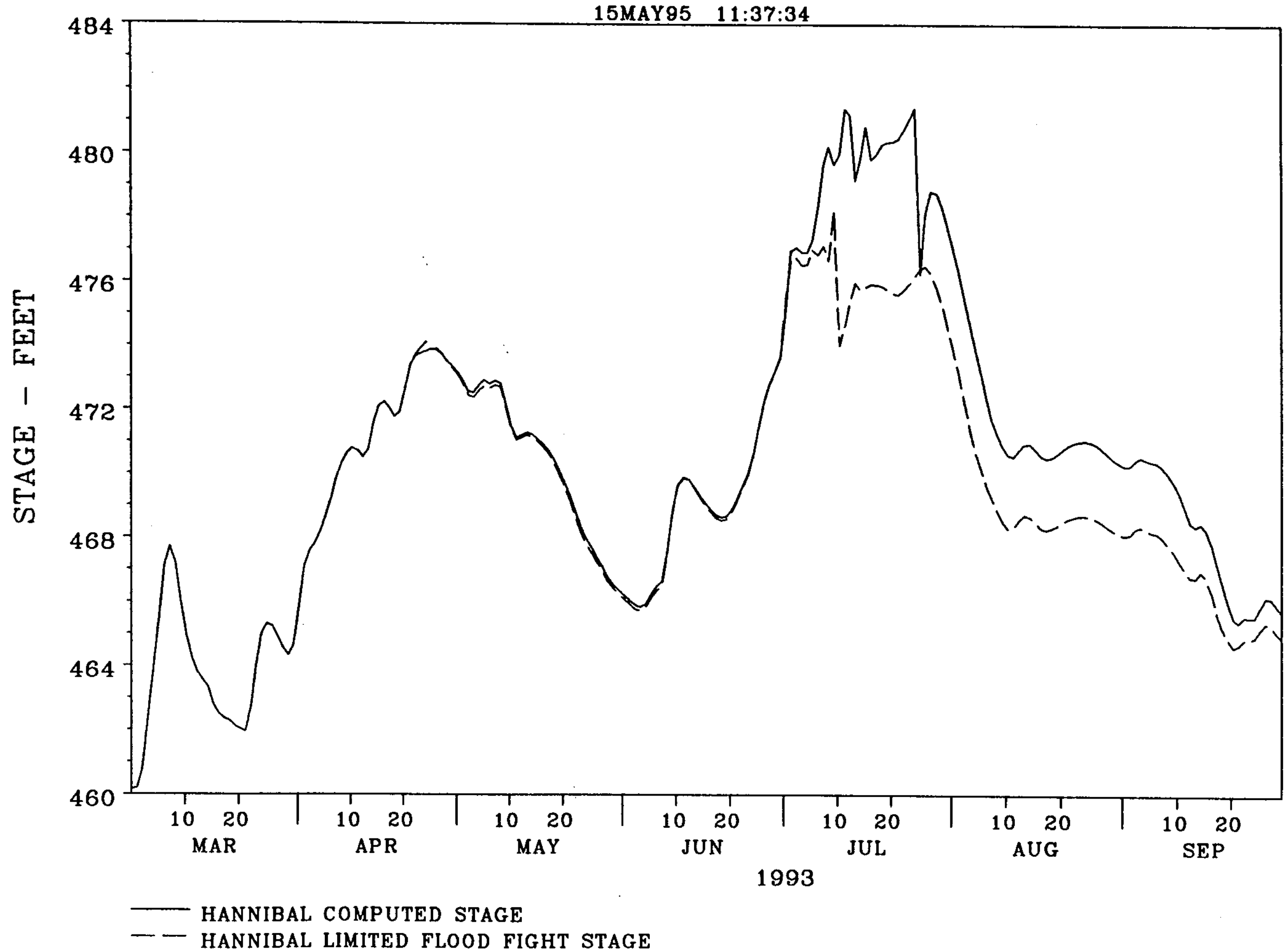
MISSISSIPPI RIVER
BURLINGTON, IOWA – R.M. 403.1
LIMITED FLOOD FIGHT

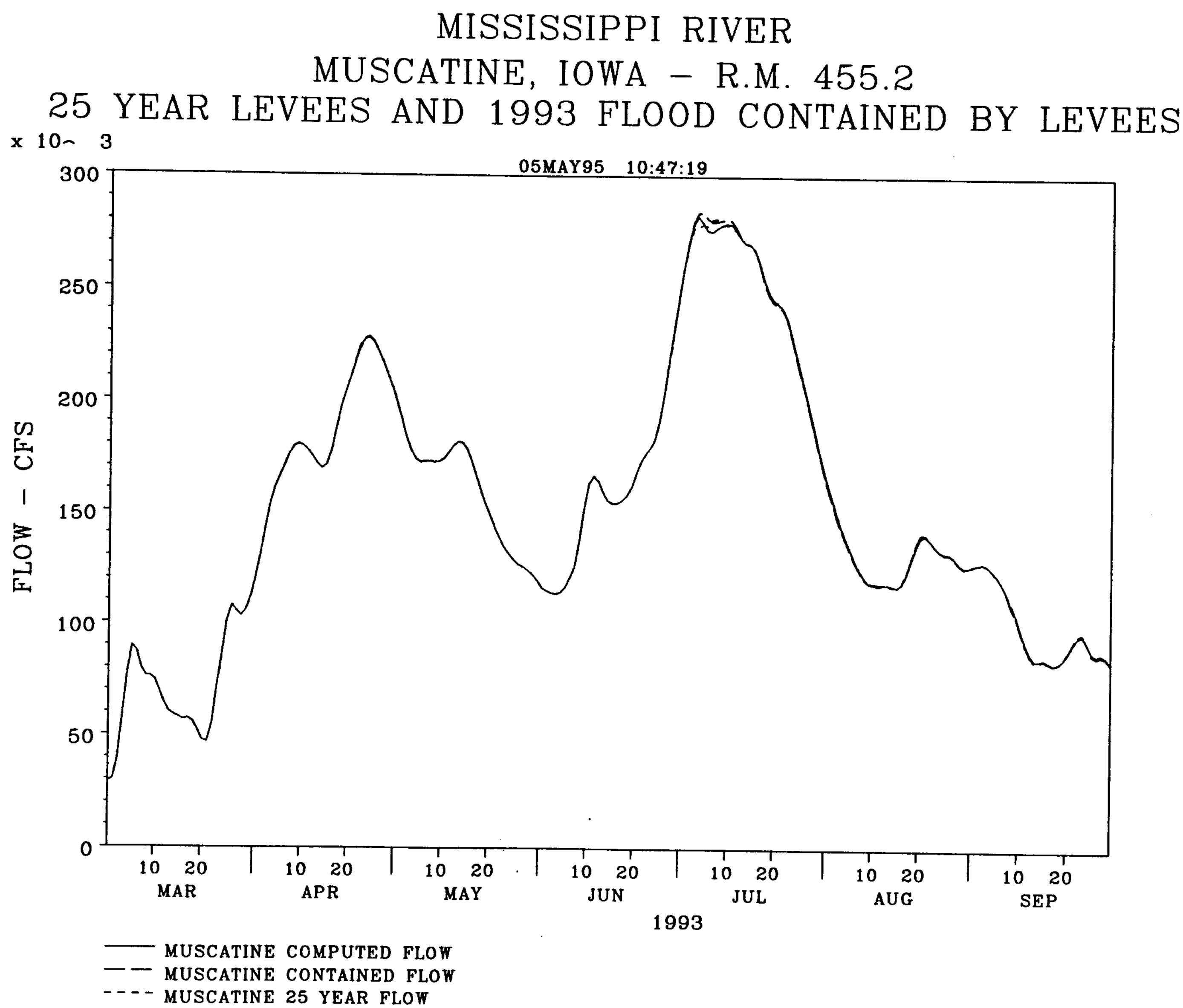


MISSISSIPPI RIVER
QUINCY, ILLINOIS – R.M. 327.9
LIMITED FLOOD FIGHT

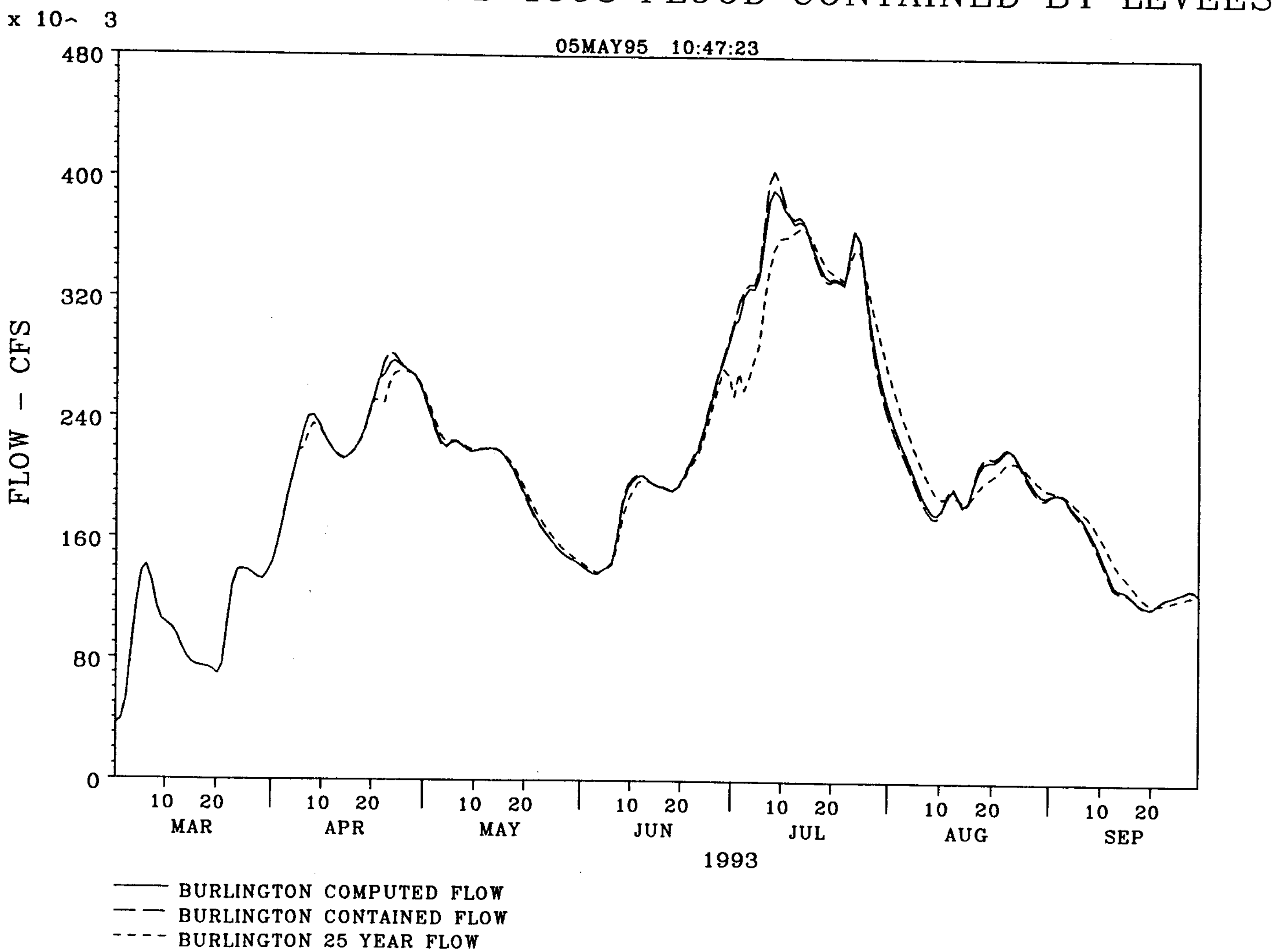


MISSISSIPPI RIVER
HANNIBAL, MISSOURI – R.M. 309.9
LIMITED FLOOD FIGHT

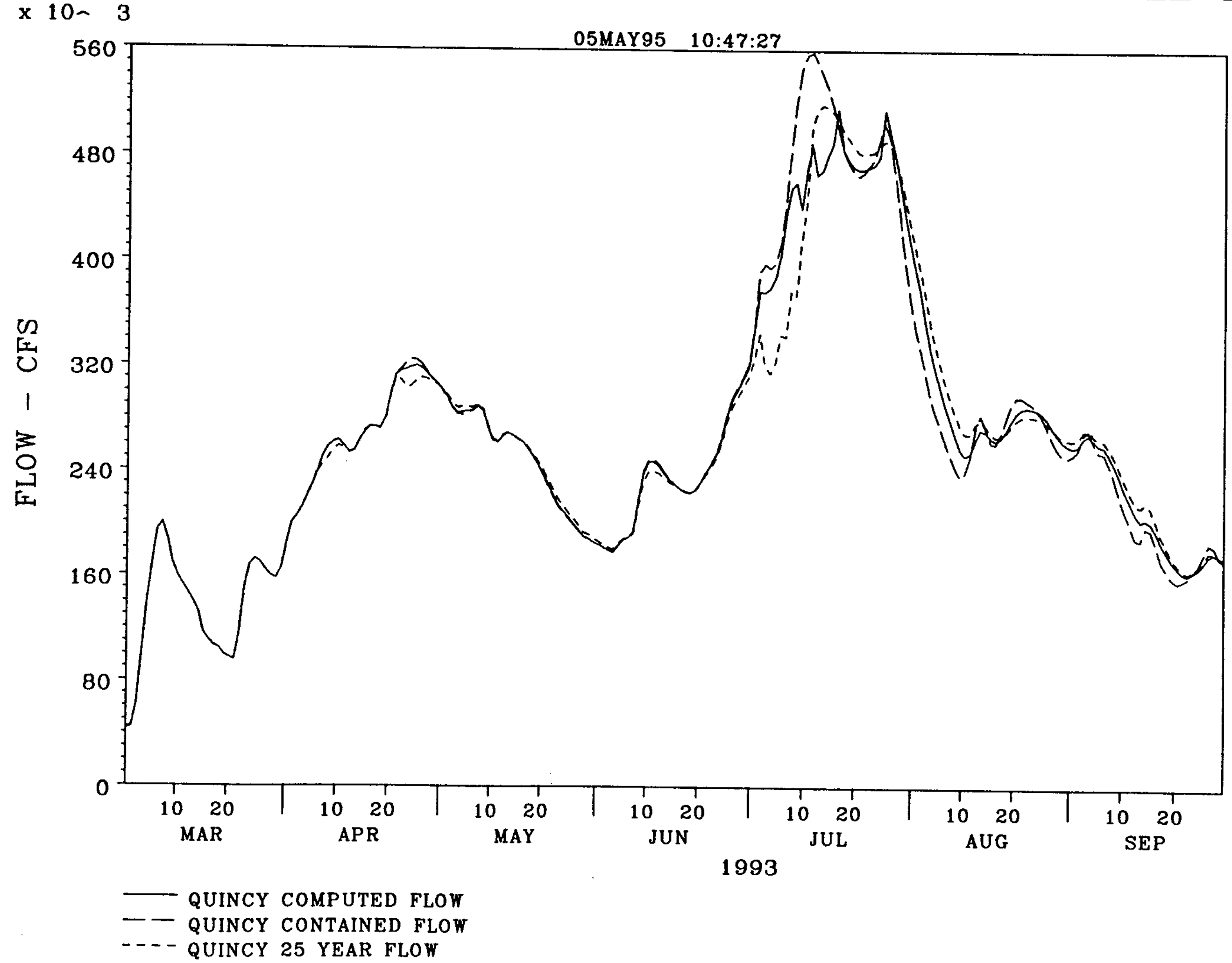


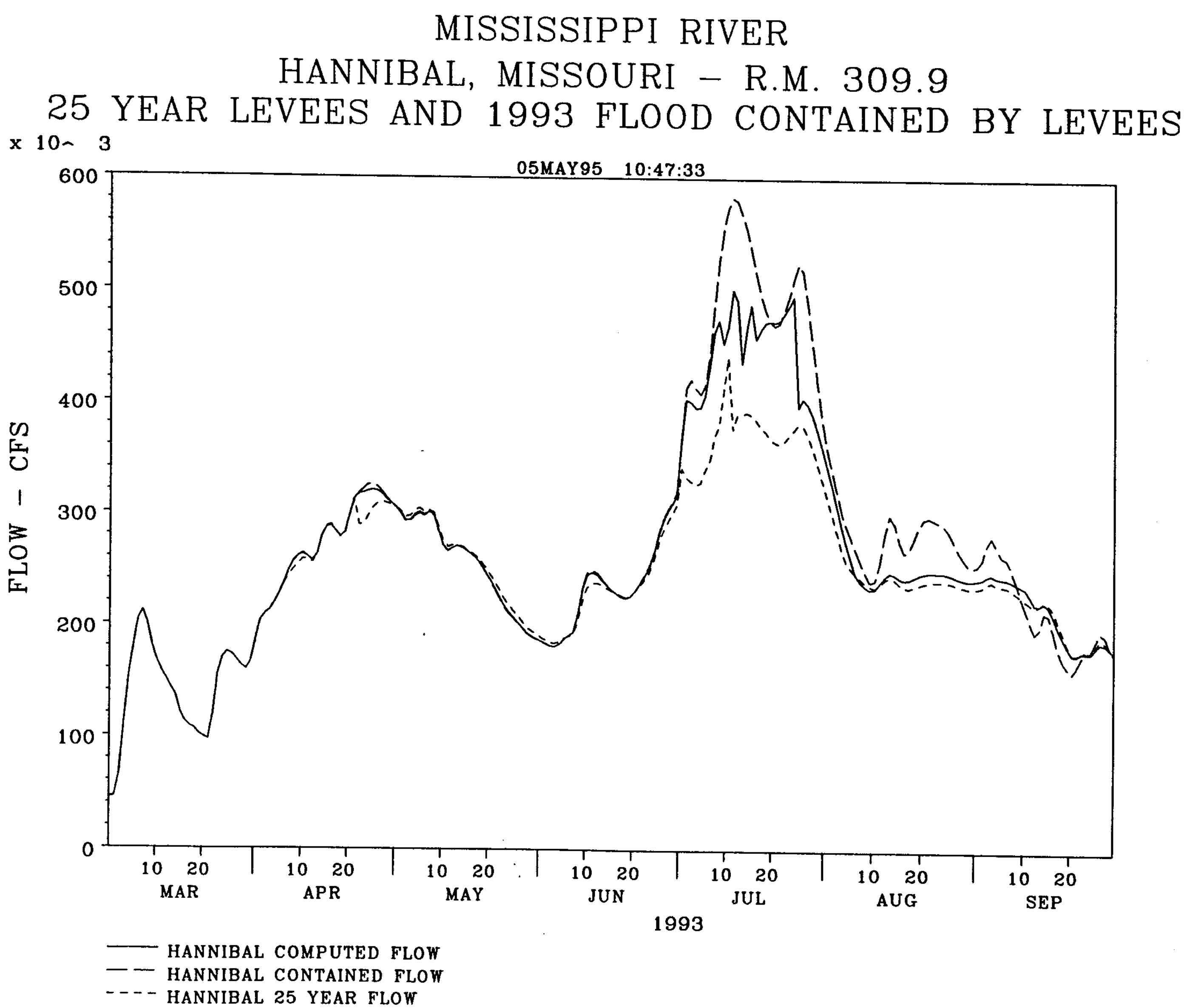

MISSISSIPPI RIVER
MUSCATINE, IOWA – R.M. 455.2
25 YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES
x 10~ 3
05MAY95 10:47:19
FLOW – CFS
0
50
100
150
200
250
300
10 20
MAR
APR
MAY
JUN
JUL
AUG
SEP
1993
MUSCATINE COMPUTED FLOW
MUSCATINE CONTAINED FLOW
MUSCATINE 25 YEAR FLOW

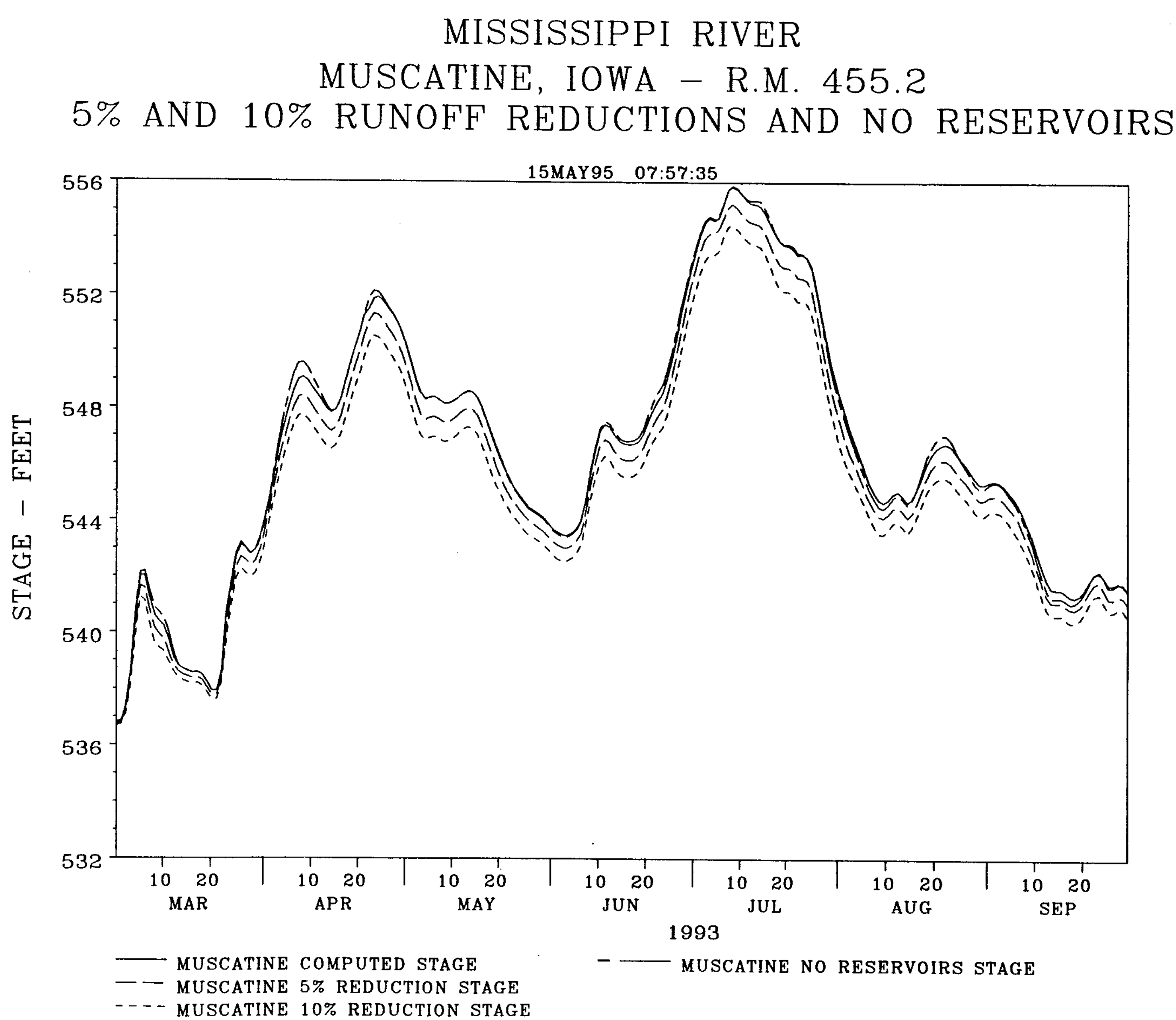
MISSISSIPPI RIVER
BURLINGTON, IOWA – R.M. 403.1
25 YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



MISSISSIPPI RIVER
QUINCY, ILLINOIS – R.M. 327.9
25 YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES




MISSISSIPPI RIVER
HANNIBAL, MISSOURI – R.M. 309.9
25 YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES
x 10~ 3
05MAY95 10:47:33
FLOW – CFS
600
500
400
300
200
100
0
10 20
MAR
10 20
APR
10 20
MAY
10 20
JUN
10 20
JUL
10 20
AUG
10 20
SEP
1993
HANNIBAL COMPUTED FLOW
HANNIBAL CONTAINED FLOW
HANNIBAL 25 YEAR FLOW


MISSISSIPPI RIVER
MUSCATINE, IOWA – R.M. 455.2
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS
15MAY95 07:57:35
STAGE – FEET
556
552
548
544
540
536
532
10 20 MAR
10 20 APR
10 20 MAY
10 20 JUN
10 20 JUL
10 20 AUG
10 20 SEP
1993
MUSCATINE COMPUTED STAGE
MUSCATINE 5% REDUCTION STAGE
MUSCATINE 10% REDUCTION STAGE
MUSCATINE NO RESERVOIRS STAGE

MISSISSIPPI RIVER
BURLINGTON, IOWA – R.M. 403.1
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS

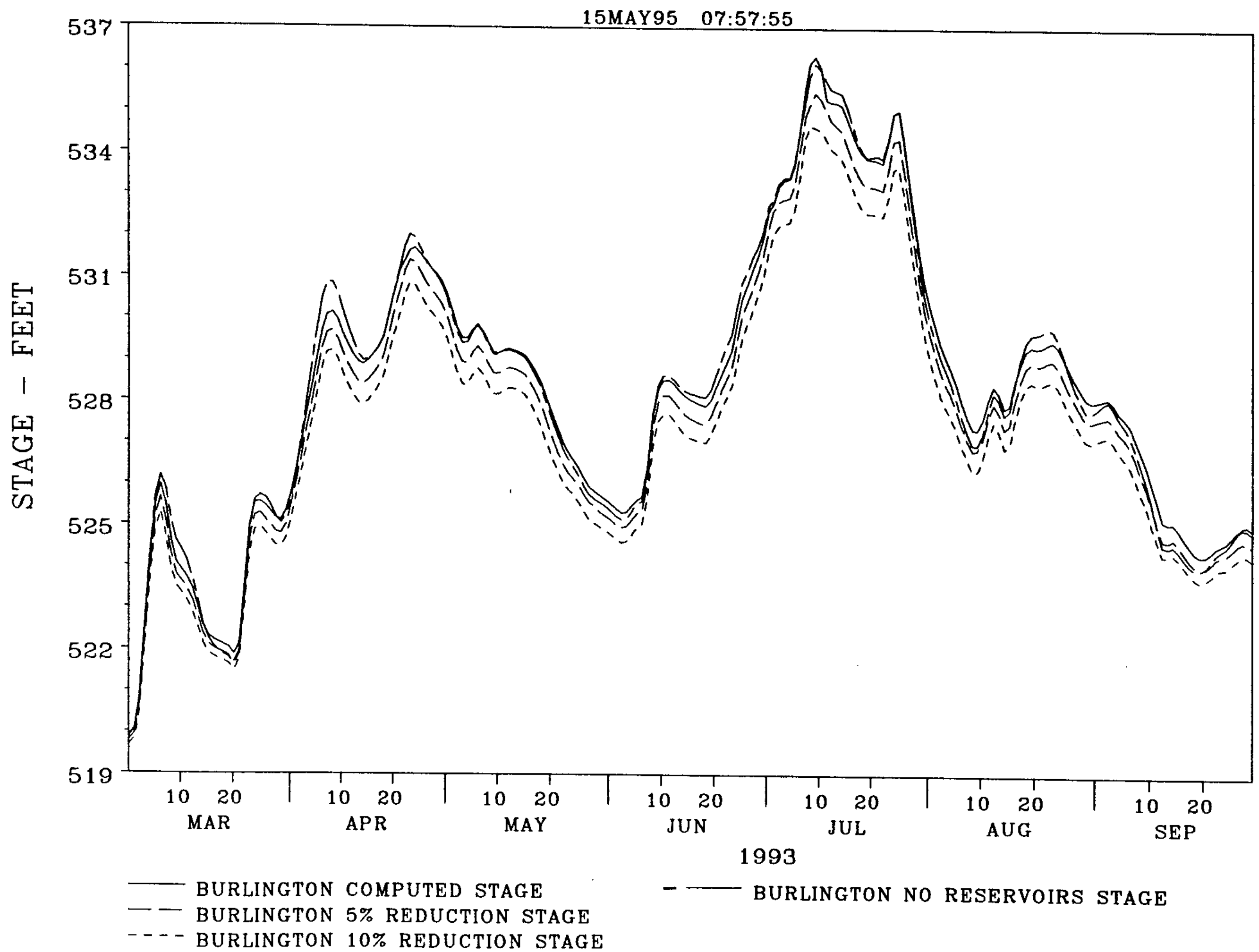
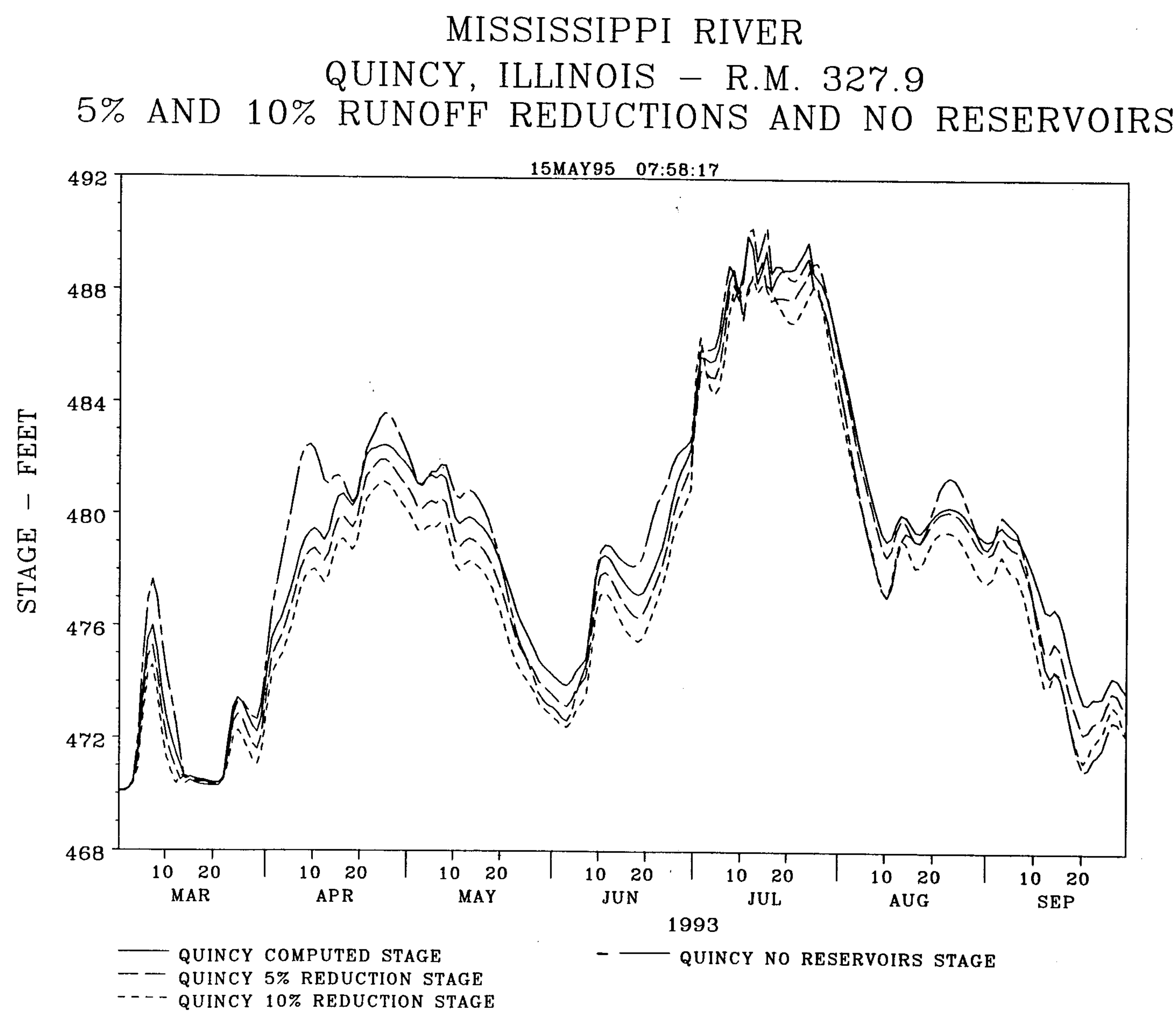


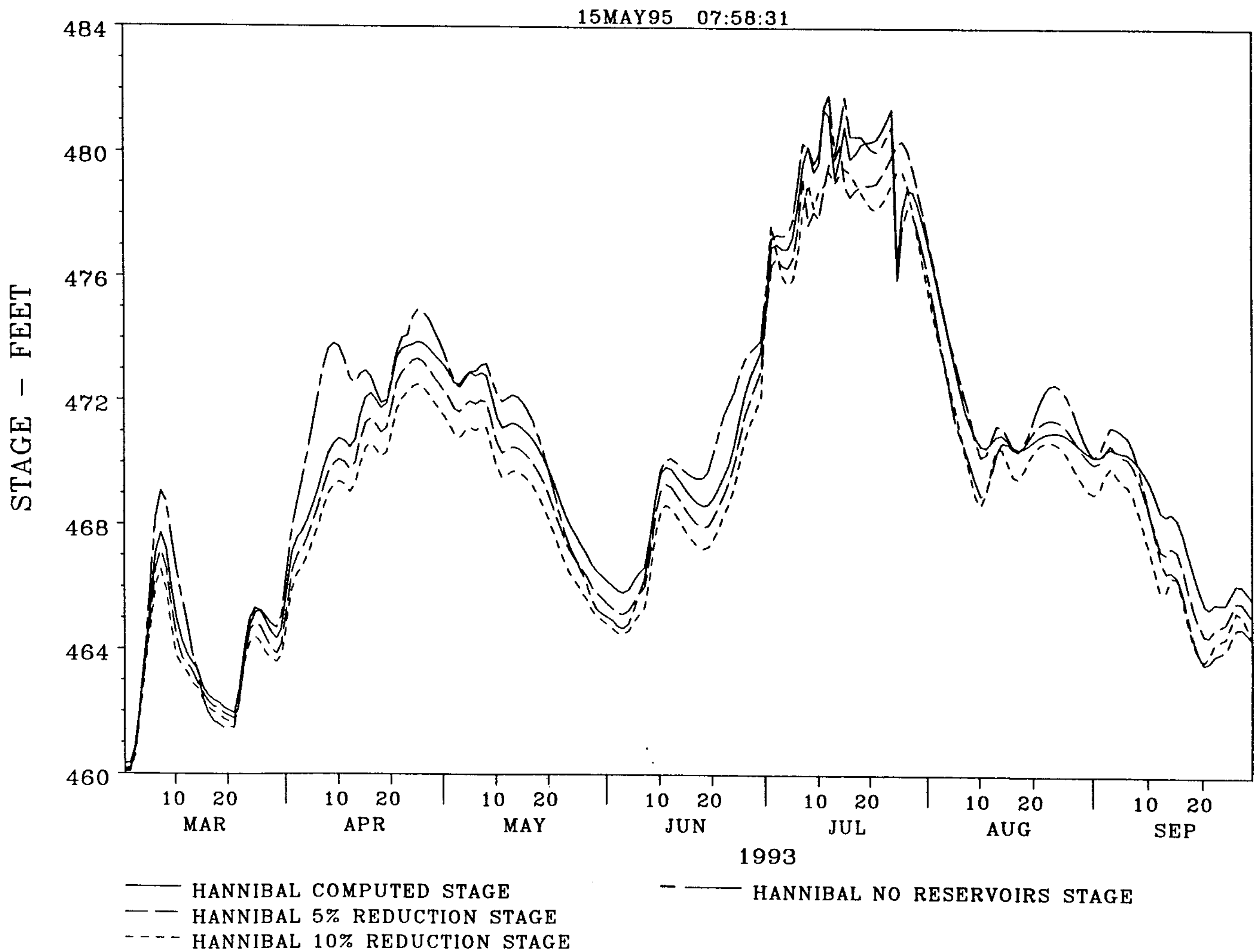
PLATE RI-35



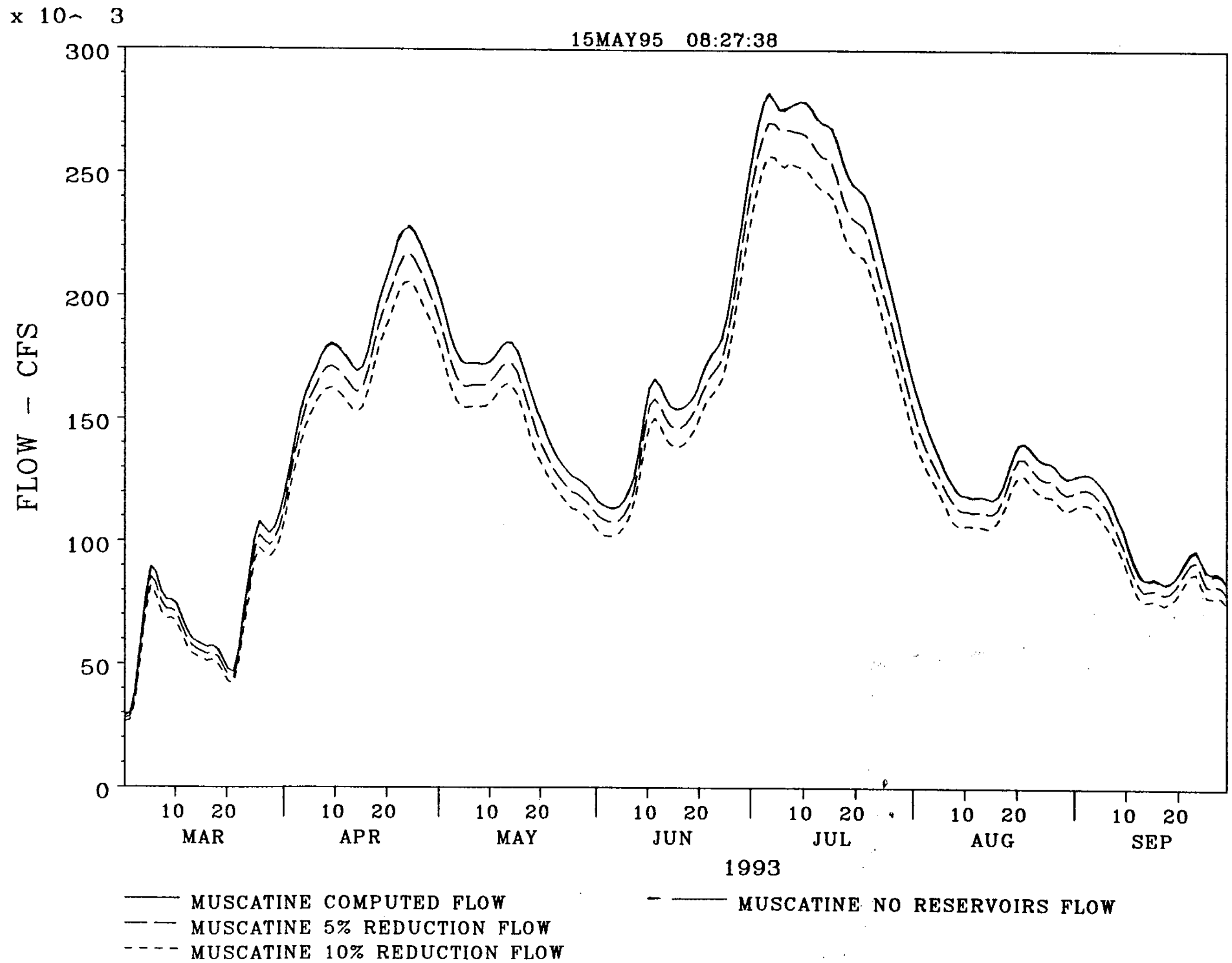
MISSISSIPPI RIVER
QUINCY, ILLINOIS – R.M. 327.9
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS
15MAY95 07:58:17
STAGE – FEET
492
488
484
480
476
472
468
10 20
MAR
10 20
APR
10 20
MAY
10 20
JUN
10 20
JUL
10 20
AUG
10 20
SEP
1993
QUINCY COMPUTED STAGE
QUINCY 5% REDUCTION STAGE
QUINCY 10% REDUCTION STAGE
QUINCY NO RESERVOIRS STAGE

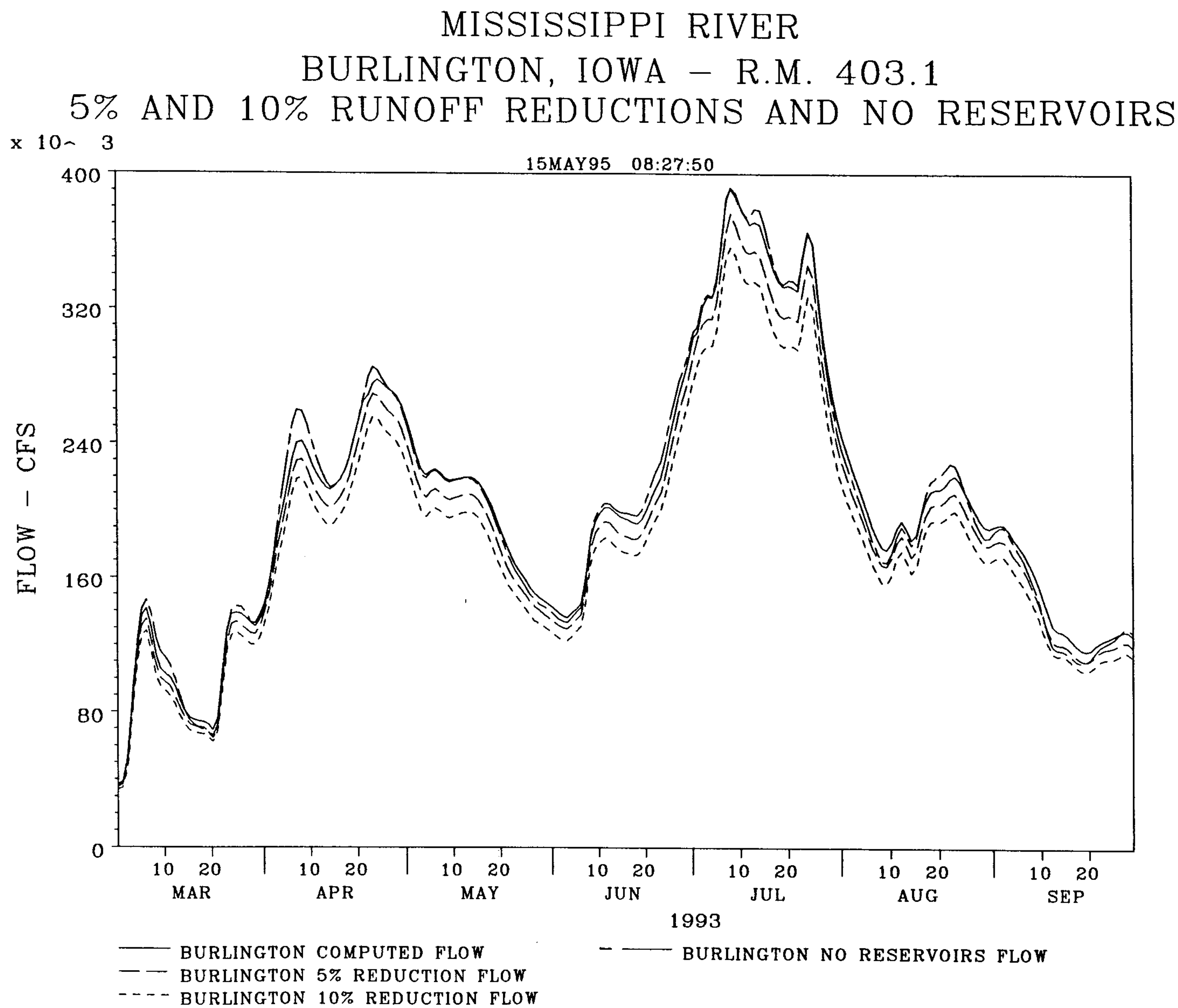
PLATE RI-36

MISSISSIPPI RIVER
HANNIBAL, MISSOURI – R.M. 309.9
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS



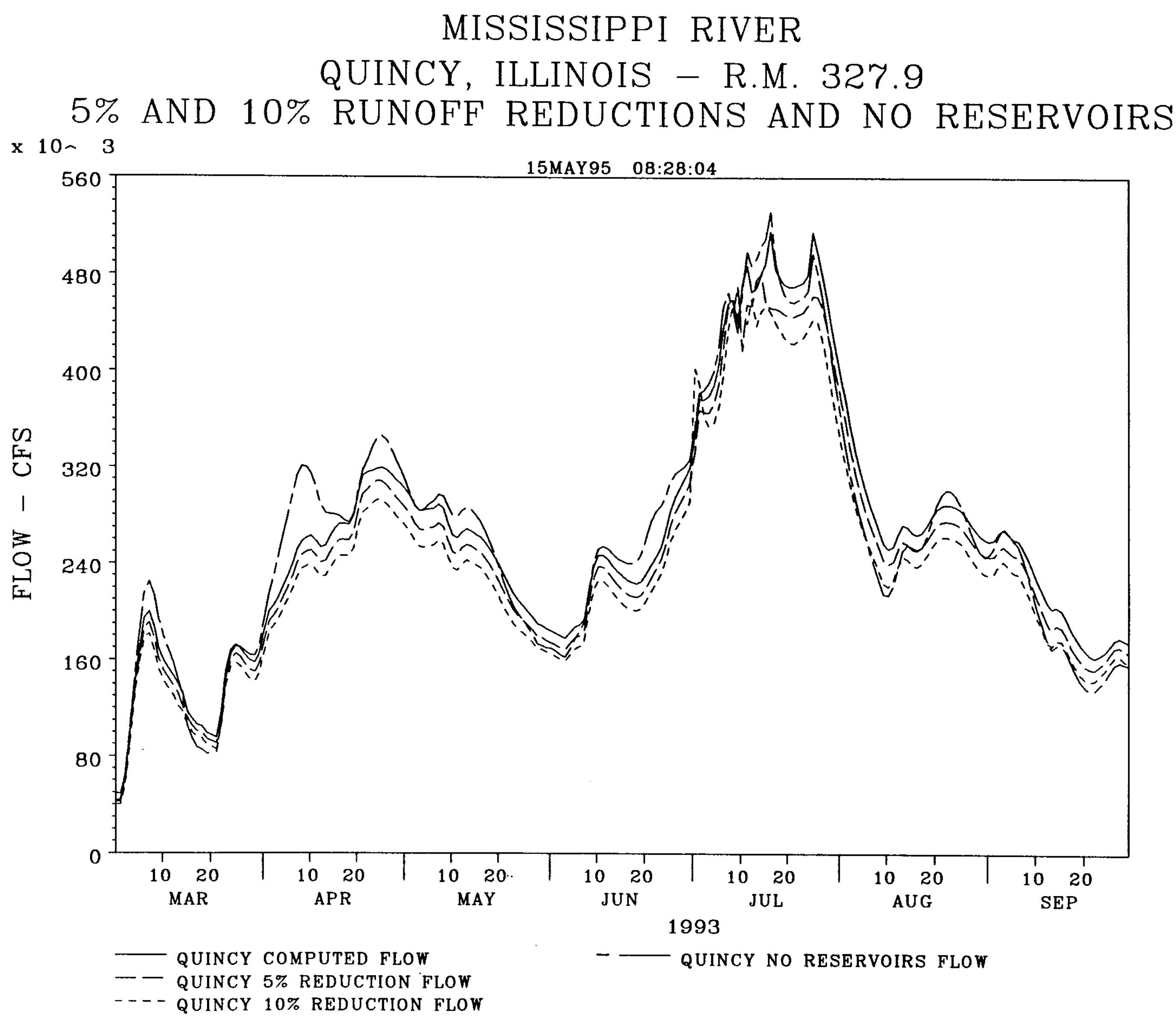
MISSISSIPPI RIVER
MUSCATINE, IOWA – R.M. 455.2
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS





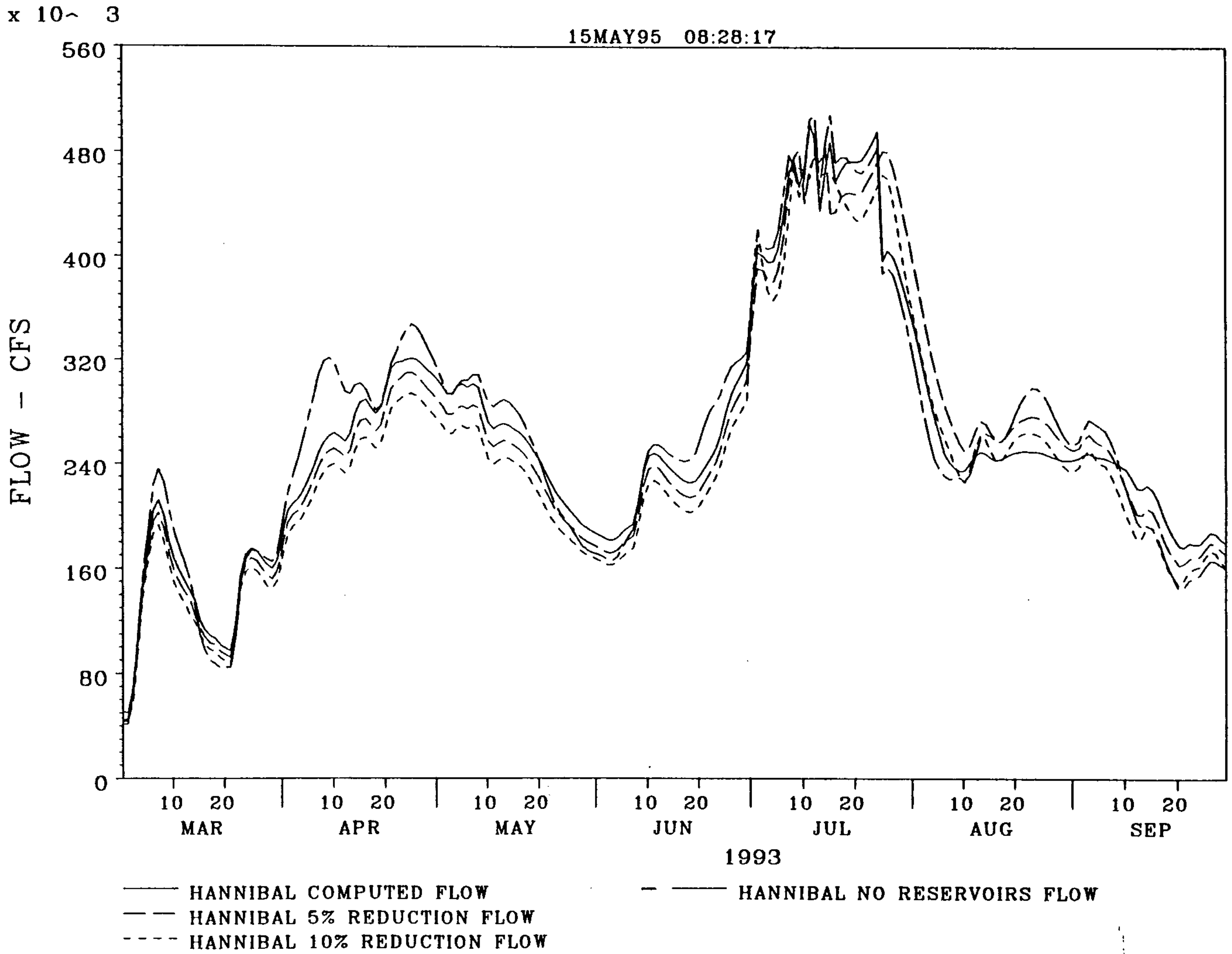
MISSISSIPPI RIVER
BURLINGTON, IOWA – R.M. 403.1
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS
x 10~ 3
15MAY95 08:27:50
FLOW – CFS
400
320
240
160
80
0
10 20 MAR
10 20 APR
10 20 MAY
10 20 JUN
10 20 JUL
10 20 AUG
10 20 SEP
1993
BURLINGTON COMPUTED FLOW
BURLINGTON 5% REDUCTION FLOW
BURLINGTON 10% REDUCTION FLOW
BURLINGTON NO RESERVOIRS FLOW

PLATE RI-39



MISSISSIPPI RIVER
QUINCY, ILLINOIS – R.M. 327.9
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS
x 10~ 3
15MAY95 08:28:04
FLOW – CFS
560
480
400
320
240
160
80
0
10 20
MAR
10 20
APR
10 20
MAY
10 20
JUN
10 20
JUL
10 20
AUG
10 20
SEP
1993
QUINCY COMPUTED FLOW
QUINCY 5% REDUCTION FLOW
QUINCY 10% REDUCTION FLOW
QUINCY NO RESERVOIRS FLOW

MISSISSIPPI RIVER
HANNIBAL, MISSOURI – R.M. 309.9
5% AND 10% RUNOFF REDUCTIONS AND NO RESERVOIRS



FPMA

FloodPlain Management Assessment

Hydrology and Hydraulics

St Louis District

May 1995
Final Report

US Army Corps
of Engineers

FLOODPLAIN MANAGEMENT ASSESSMENT
U.S. Army Corps od Engineers
St. Louis District
Hydraulics and Hydrology
Appendix A

Table of Contents

Table of Contents Continued

List of Tables

NOTE: Tables SL-2 through SL-12 are for all alternatives except setback.
Tables SL-13 through SL-23 are for setback alternatives only.

List of Plates

NOTE: The following notations are used for X:

1 COMPUTED VS OBSERVED STAGES
2 5% & 10% RUNOFF REDUCTION AND NO RESERVOIR
3 25-YEAR LEVEE AND 1993 FLOOD CONTAINED
4 LEVEES REMOVED: AGRICULTURAL OR NATURAL
5 SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS

FLOOD PLAIN MANAGEMENT ASSESSMENT
U.S. Army Corps of Engineers
St. Louis District

Hydraulics and Hydrology
Appendix A

1. Study Purpose

The Flood Plain Management Assessment was initiated by Congress for a comprehensive study of the Missouri and Mississippi Rivers following the 1993 flood event. The role of the hydraulic and hydrologic team was to construct a modeling tool capable of assessing current floodplain policy and construction impacts on a system wide basin. To accomplish this task, an unsteady flow model (UNET) was constructed of the Mississippi, Missouri, and significant tributary rivers. Corps District offices along the Mississippi River include St. Paul, Rock Island, and St. Louis, and along the Missouri River include Omaha and Kansas City. The Districts developed a systemic hydraulic modeling effect. While coordinating with all involved Corps Districts, each unsteady flow model was developed independently by each District. Assimilation of model results and system-wide routing was then performed for all conditions examined.

2. Study Area

The Mississippi River rises in the lake forest country of north central Minnesota near the Village of Bemidji, and flows north, east and then south through this timbered landscape to Minneapolis-St. Paul. At this point, it leaves the northern woodlands and lakes, and meanders southward past fertile prairies and many villages and cities. Along the way, tributaries that drain lands to the east and to the west join the Mississippi River and add to its flow. The Mississippi River flows 1,370 miles from its headwaters to the confluence with the Ohio River. Then it flows another 964 miles to the Gulf of Mexico. The boundary between the North Central and Lower Mississippi Valley Divisions is located about nine miles downstream from Hannibal, Missouri, just downstream of Lock and Dam 22. The drainage area upstream from this boundary is about 137,500 square miles. The St. Louis District is one of four districts that make up the Lower Mississippi Valley Division. The District covers 27,000 square miles. The major tributaries of the Mississippi River in the St. Louis District boundary are as follows:

a. Salt River Basin

The Salt River Basin lies in northeastern Missouri and has a drainage area of approximately 2,934 square miles. One multiple purpose reservoir, Mark Twain Lake (Clarence Cannon Dam), has been constructed approximately 63 miles above the confluence of the Salt River and the Mississippi River. The watershed for the reservoir is 2,314 square miles, or about 79 percent of the total Salt River Basin. Along the main stream below the dam, the bottomlands vary from 2,000 feet to more than a mile in width.

b. Illinois River Basin

The Illinois River Basin extends southwesterly across the northern half of the State of Illinois from Chicago to the Mississippi River at Grafton, Illinois, 38.4 miles above St. Louis, Missouri. It extends northerly to just west of Milwaukee, Wisconsin, and easterly to South Bend, Indiana. The total natural drainage area is about 28,906 square miles, with about 1,000 square miles in Wisconsin, 3,000 square miles in Indiana and 25,000 square miles in Illinois.

The Illinois River is the largest tributary of the Mississippi River Basin above the mouth of the Missouri River. It lies entirely within the boundaries of the State of Illinois and is formed by the confluence of the Kankakee and Des Plaines Rivers about midway between Chicago and La Salle, Illinois. From this point, the Illinois River flows in a southwesterly direction 273 miles to the Mississippi River. The average water surface slope through the St. Louis District reach is small with little or no evidence of erosion along the banks and streambed. Throughout the greater part of the lower 80 miles, the river follows the base of the western bluff.

c. Missouri River Basin

The Missouri River basin is the biggest of the nation's 18 major water resource regions, embracing 529,400 square miles within the United States, including all or parts of 10 states. The basin also includes 9,715 square miles in Canada. On the west are the Rocky Mountains, and on the east are the productive farmlands of the Missouri-Mississippi drainage areas. The Missouri River flows in a southeasterly direction 2,315 miles from its headwaters at Three Forks, Montana, through six major main stem reservoirs to its confluence with the Mississippi about 15 miles above St. Louis, Missouri.

d. Meramec River Basin

The Meramec River Basin lies in the east central portion of the State of Missouri. The basin is bounded on the north by the Missouri River Basin; on the east by the Mississippi River; on the south by the St. Francis, Black, and Current Rivers; and on the west by the Gasconade River Basin. The drainage area of the entire watershed is about 3,980 square miles of which 3,788 square miles are upstream of the Eureka, Missouri stream gage. The watershed comprises all or portions of 15 counties and converges toward the City of St. Louis. The drainage system consists of the Meramec River and its two principle tributaries, the Big River and the Bourbeuse River.

e. Kaskaskia River Basin

The Kaskaskia River Basin lies entirely in the State of Illinois. It is the second largest in the State of Illinois with a drainage area of 5,801 square miles. It extends from the center of Champaign County in the southwesterly direction to the Mississippi River near the City of Chester. Elevations range from about 730 feet at the headwaters to about 385 feet at the bluff line where it emerges into the Mississippi River flood plain. The basin has a median length of 175 miles, a maximum width of 55 miles and an average width of 33 miles. The topography of the basin is generally flat or gently rolling except for broken terrain near the streams.

The Kaskaskia River rises in Champaign County, Illinois about 5 miles northwest of Urbana, in the east-central part of the State. It flows southwesterly for about 310 miles and empties into the Mississippi River above Chester, Illinois, 118 miles above the mouth of the Ohio River. Multiple purpose reservoirs were constructed at river mile 106.6 (Carlyle Lake) and river mile 221.8 (Lake Shelbyville).

3. Navigation/Reservoir Structures

As one of the four districts that make up the Lower Mississippi Valley Division, the St. Louis District is responsible for administering Federal water resource development programs in large portions of Missouri and Illinois.

The District operates and maintains four locks and dams on its portion of the mainstem of the Mississippi River: Lock and Dam 24, Lock and Dam 25, Melvin Price Locks and Dam, Locks 27; and one lock and dam on the Kaskaskia River. It operates three multi-purpose reservoirs in Illinois: Lake Shelbyville and Carlyle Lake on the Kaskaskia River, and Rend Lake on the Big Muddy River. It operates Mark Twain Lake on the Salt River, and Wappapello Lake on the St. Francis River, in Missouri. The St. Francis River will not be addressed in this report because it enters the Mississippi River downstream of the Ohio River.

a. Mark Twain Lake

Mark Twain Lake is located on the Salt River about 63 miles upstream from the confluence with the Mississippi River. The authorized purposes of this project include flood control, hydroelectric power generation, water supply, fish and wildlife conservation, recreation, and water quality enhancement. Incidental navigation benefits on the Mississippi River occur as the result of releases from the lake during low-flow periods. The Mark Twain Lake watershed is comprised of 2,318 square miles, with an additional 29 square miles draining into the Reregulation Pool. The joint-use pool at the dam is between 567.2-606.0 feet National Geodetic Vertical Datum (NGVD), with 457,000 acre-feet of conservation storage. The flood-control pool at the dam is between 606.0-624.8 feet National Geodetic Vertical Datum (NGVD), with 442,000 acre-feet of flood-control storage.

b. Lake Shelbyville

Lake Shelbyville is a multi-purpose reservoir located on the Kaskaskia River, one-half mile east and one-fourth mile north of the Town of Shelbyville, Illinois. The purposes of this reservoir are to provide flood protection, to create recreational opportunities, to provide fish and wildlife conservation, to augment water supplies, to enhance water quality, and to augment flows for navigation. The drainage area of the lake is 1,054 square miles. The joint-use pool is between elevations 573.0-599.7 feet NGVD with 180,000 acre-feet of conservation storage. The flood-control pool is between elevations 599.7-626.5 feet NGVD with 474,000 acre-feet of flood-control storage.

c. Carlyle Lake

Carlyle Lake is a multi-purpose reservoir on the Kaskaskia River extending from the main dam at river mile 106.6 near Carlyle, Illinois, to river mile 160.0. The purposes of this project are to provide flood protection; to create recreational opportunities; to augment water supplies; to enhance water quality; to augment flows for navigation on the Kaskaskia River; and to provide fish and wildlife conservation.

The drainage area of the lake is 2,717 square miles. The joint-use pool is between elevations 429.5-445.0 feet NGVD with 233,000 acre-feet of conservation storage. The flood-control pool is between elevations 455.0-462.5 feet NGVD with 699,900 acre-feet of flood-control storage.

d. Rend Lake

Rend Lake is a multi-purpose reservoir located on the upper reaches of the Big Muddy River in southern Illinois. The dam is located on the Big Muddy River at River Mile 103.7 about 3 miles northwest of Benton, Illinois, and the reservoir at normal levels extends upstream approximately 13 miles. The purposes of this project are to provide flood protection; to create recreational opportunities; to augment water supplies; to enhance water quality; and to provide fish and wildlife conservation. The drainage area of the lake is 488 square miles. The joint-use pool is between elevations 391.3-405.0 feet NGVD with 160,000 acre-feet of conservation storage. The flood-control pool is between elevations 405.0-410.0 feet NGVD with 109,000 acre-feet of flood-control storage. The flood control pool is limited to protecting the area downstream of the dam to floods not exceeding a 5-year frequency of occurrence. Two subimpoundments, one each on the Big Muddy and Casey Fork arms, are located in the upper reaches of the reservoir. They consist of compacted earth core and rock embankment with the crest elevation at 416.0 and overflow section at elevation 412.0.

4. Synopsis of the Flood

The Great Flood of 1993 affected a large portion of the midwestern United States, crossing the boundaries of several Corps of Engineers Districts, including St. Paul (CENCS), Chicago (CENCC), Rock Island (CENCR), Omaha (CEMRO), Kansas City (CEMRK), and St. Louis (CELMS). Each of these districts experienced some degree of flooding during the spring and summer 1993. The St. Louis District experienced extensive flooding.

The flooding on the Mississippi River was the most devastating, in terms of property and disrupted business, in the history of the United States. Millions of acres of farmland were under water for weeks during the growing season. Damaged highways and roads disrupted overland transportation throughout the flooded region. The river was closed to navigation for several weeks. The banks and channels of the Mississippi River were severely eroded in many reaches. In addition to the erosion of the river, erosion of valuable topsoil was a major problem. The extent and duration of the flooding caused numerous levees to overtop or breach. Every gaging station on the Mississippi River within the St. Louis District, experienced a new stage of record. Specific details of the flood can be found in "The Great Flood of 1993 Post-Flood Report", Upper Mississippi River Basin, Appendix C, September 1994.

5. Levee Design

In the St. Louis District, federal levees and floodwalls protected over 250,000 acres from flooding in 1993. There are 39 federal and numerous non-federal and private levee districts within the St. Louis District boundary to provide partial flood protection to specific areas along the Mississippi River.

a. Federal Levees

Federal levees are those built and maintained by the Federal government. The standard Federal levees are designed for a 20-500 year+ flood event. The Mississippi River Federal agricultural levee

systems, (Columbia, Harrisonville, Ft. Chartres, Stringtown, Prairie du Rocher, Kaskaskia Island, Bois Brule, Degogonia, Grand Tower, Preston, Clear Creek and East Cape Drainage and Levee districts) are designed to the 50-year level of protection. The Illinois River Federal levee systems (Nutwood, Eldred and Spanky, Keach, Hartwell, Hillview, Big Swan, Scott County, Valley City, Mauvaise Terre, Willow Creek and McGee Creek Drainage and Levee districts) are designed to varying levels of protection from 20-100+ year. The urban Federal levee systems, (Wood River, East St. Louis, City of St. Louis, and Prairie du Pont Drainage and Levee districts) are designed to the 500 year level of protection. The Cape Girardeau Federal urban levee is designed to the 200-year level of protection. Although some Federal levees are constructed with a minimum five-foot-thick clay cap, most Federal levees in St. Louis District are constructed entirely of clay and have side slopes of 1V:3-4H. Crown widths on Federal levees vary from 10 to 20 feet. Federal levees also include seepage control measures where determined necessary. During the Flood of 1993, 39 Federal levees breached/overtopped out of a total of 229 along the Upper Mississippi-Missouri River system. Nearly all the Federal levees overtopped or breached at river elevations greater than levee design. Thus, these losses of Federal levees should be properly classified as "design exceedances" and not "failures".

b. Non-Federal and Private Levees

Non-Federal levees are levees constructed with other than Federal funds by an organized levee district. Private levees are levees constructed, owned and maintained by one or more individual landowners. The typical private levee is designed for a 5-20 year flood event. They are generally constructed of a mixture of soil materials with side slopes of 1V:2H. Private levees often have narrow crown widths of 4-5 feet and little or no seepage control. Private levees are also typically located closer to the river than federal levees. During the Flood of 1993, about 80 percent of the private levees were overtopped or breached. Because of the relatively frequent design flood, this high loss is not surprising. Protected areas are primarily cropland and the resulting flood damages generally cannot justify higher levels of protection.

Three private levees on the Missouri River are considered to be private urban levee systems. Riverport and Earth City levee districts are designed to a 500-year level of protection. The remaining Chesterfield-Monarch levee district is designed to a 100-year level of protection. The Chesterfield-Monarch levee breached during the 1993 flood.

6. Unsteady Flow Model

A one-dimensional unsteady flow hydraulic model was developed for the St. Louis District for a portion of the Mississippi River and its associated floodplains to test the effects of the current levee system on the flood of 1993. The analysis was conducted using the UNET program (Barkau, 1993) currently supported by Dr. Robert Barkau in conjunction with the Hydrologic Engineering Center, US Army Corps of Engineers. For the purposes of the FPMA, the UNET program was further developed by Dr. Robert Barkau. Program changes include enhancements to the levee filling algorithm, calibration methods, and conveyance with the overbank. The UNET program is an unsteady flow model that is suited for modeling long reaches of rivers where the dynamic effects of levee breaches, backwater conditions, bed slopes of less than one foot per mile, and varying flow rates along the river are important. All these effects are important for understanding the processes of the 1993 flood and the effects of levees on floods.

Specific data are necessary to develop the UNET model. These data requirements are used to simulate river geometry and flow conditions:

a. river cross-sections,
b. Manning's "n" values (roughness coefficients)
c. observed flow and stage data for model calibration, and
d. boundary conditions of flow and stage.

All these data were available or derived for the actual flood condition for which the model was calibrated.

The St. Louis District UNET model was developed for the reach of the Mississippi River from near Hannibal, MO (Lock and Dam 22 tailwater) to Cairo, IL; the Missouri River form Hermann, MO (R.M. 97.9) to the mouth; the Illinois River from near Meredosia, IL (R.M. 70.8) to the mouth; and the Kaskaskia, Meramec, and Salt Rivers from the mouth upstream to the first river gaging station. All major tributaries entering the Mississippi River are modeled from the last rated gage downstream to the mouth to reproduce the effects of backwater on the outflows. A graphical schematic of this UNET model is depicted in Plate SL-1.

7. Systemic Approach for the UNET Models

To accomplish a systemic approach, hydrologic/hydraulic results from the UNET models were transferred at specific stream locations between Corps districts. These locations were selected using the following criteria: 1) district boundaries, 2) federal/private levee district boundaries, 3) backwater condition and 4) cross section geometry. Cross sectional data for the upstream UNET model overlapped the downstream UNET model to eliminate the influence of downstream boundary conditions of the upstream UNET model. For the Missouri River, the Omaha District modeled from Omaha, NE (R.M. 615.97) downstream to R.M. 410.0 with the transferred location at the St. Joseph, MO gage (R.M. 448.2), the Kansas City District modeled from St. Joseph, MO gage to St. Charles, MO gage (R.M. 28.2) with the transferred location at Hermann, MO gage (R.M. 97.9), and the St. Louis District from Hermann, Mo gage to the mouth. For the Mississippi River, the Rock Island district modeled from Lock and Dam 10 (R.M. 615.1) to Grafton, MO gage (R.M. 218.0) with the transferred location at Lock and Dam 22 TW (R.M. 301.1), and the St. Louis District modeled from Lock and Dam 22 TW to Cairo, Il (R.M. 0.0).

8. Model Calibration-Base Condition

The UNET model was calibrated to the 1973, 1986 and 1993 floods. The 1973 and 1986 floods were evaluated as a means to estimate the effects of the levees for lesser flood events. The 1973 event varied from a 20-100+ year flood on the Mississippi River. The 1993 flood corresponded to a 30-year event on the Mississippi River at the St. Louis gage. The 1986 flood event was about a 50-year event on the Missouri River and about a 10-year event on the Mississippi River above the confluence with the Missouri. The 1993 flood event was about a 175-year event at the St. Louis, MO gage and varied in frequency on the Missouri River and Mississippi River.

The initial UNET calibration of the St. Louis model was performed by Dr. Barkau for the Scientific Assessment and Strategy Team (SAST). The results of his analysis can be found in "A BLUEPRINT FOR CHANGE PART V", Science for Floodplain Management into the 21st Century, Scientific

Assessment and Strategy Team, Washington, D.C., June 1994.

Additional cross sections, levee definitions and calibrations modified the St. Louis UNET model. This calibrated model is considered the base condition for all comparisons in this report. The systemic results for the 1993 flood are displayed in Table SL-1. The average peak stage error (observed minus computed) displayed in this table is 0.4 feet. Computed and observed stage hydrographs are shown on Plates SL-2 through SL-14 with the suffix of .1. The levees performance on the Mississippi River, Illinois River and Missouri River during the 1993 flood are displayed in Tables SL-5 through SL-9.

9. Agricultural Levee Alternatives

The effect of several alternative agricultural levee height and locations were analyzed employing the calibrated UNET model developed for the base condition. For each alternative, the base condition UNET model was modified to reflect geometry changes required to simulate the effect on conveyance/storage within the model. Calibration parameters determined in the base condition were not altered for any of the alternatives. In reality, the alternatives alter conveyance within a cross section by changing effective flow area, land use, sediment deposition, and other factors.

a. No Agricultural Levees with Agricultural Growth

For this alternative, all agricultural levees were removed. The simulation was performed with agricultural growth within the overbank area. Factors affecting conveyance were not evaluated in detail. For example, removal of the levee would not result in an effective flow width equal to the entire valley width. Physical factors such as channel meandering, vegetation, topography, structures such as roads and railroads, and other components will restrict effective flow width to a value much less than the cross section width. Various forms of land use within the overbank such as farming habitat will have considerably different roughness values. Levee removal will remove channel constraints such that channel meandering and overbank sediment deposition may actually reduce conveyance.

The systemic results for this alternative at the stream gages are displayed in Tables SL-2 through SL-4. The average peak stage reduction from Lock and Dam 22 to Lock and Dam 26 is 2.1 feet, and from the Chain of Rocks (R.M. 199.9) gage to Cape Girardeau, MO gage is 4.3 feet on the Mississippi River. The average reduction in stage on the Illinois River is 1.8 feet and on the Missouri River is 0.9 feet. The change in the hydrographs because of this alternative is shown on Plates SL-2 through SL-14 with the suffix of .2. The levees removed on the Mississippi River, Illinois River and Missouri River are displayed in Tables SL-5 through SL-9. The percent of change from the computed base and alternative peak discharge for each gage site is shown in Tables SL-10 through SL-13.

b. No Agricultural Levees with Natural Growth

For this alternative, all agricultural levees were removed and the overbanks were replaced with natural growth. This natural growth would include combination of woodlands, heavy vegetation, and wetlands.

The systemic results for this alternative of removing levees with natural growth are displayed in Tables SL-2 through SL-4. The average peak stage increase from Lock and Dam 22 to Lock and Dam 26 is 0.3 feet, and the average stage increase from the Chain of Rocks gage to Cape Girardeau, MO gage

is 0.1 feet on the Mississippi River. The average increase in stage on the Illinois River is 0.8 feet and on the Missouri River is 2.4 feet. The change in the hydrographs because of this alternative is shown on Plates SL-2 through SL-14 with the suffix of .2. The percent of change from the computed base and alternative peak discharge for each gage site is shown in Tables SL-10 through SL-13.

c. No Breaching or Overtopping of any Levees

For this alternative, all agricultural and urban levees were raised so no breaching or overtopping of any levees would occur during the simulated 1993 flood. Levee locations or roughness values were not altered for this alternative.

The systemic results for this alternative of containing the 1993 flood are displayed in Tables SL-2 through SL-4. The average peak stage increase from Lock and Dam 22 to Lock and Dam 26 is 4.4 feet, and from the Chain of Rocks gage to Cape Girardeau, MO gage is 6.5 feet on the Mississippi River. The average increase in stage on the Illinois River is 5.4 feet and on the Missouri River is 5.2 feet. The change in the hydrographs because of this alternative is shown on Plates SL-2 through SL-14 with the suffix of .3. The levees raised to contain the 1993 flood are displayed in Tables SL-5 through SL-9. The percent of change from the computed base and alternative peak discharge for each gage site is shown in Tables SL-10 through SL-13.

d. Levee Height at 25-Year Level

For this alternative, the height of all agricultural levees was set to correspond with a 4% annual chance (25-year) flood. Federal levees, which are currently higher than the 25-year elevation, were notched to an elevation equal to the 25-year elevation at the downstream end of the levee. Levees lower than the 25-year were raised to the 25-year elevation plus three feet with a notch at the downstream end of the levee at the 25-year. When flood levels exceed the 25-year level, the levee notch is eroded and the cell fills with water. In this manner, the levee cells along the channel act as detention basins to store flows which exceed the 25-year event.

The systemic results for this 25-year levee alternative are displayed in Tables SL-2 through SL-4. The average peak stage decrease from Lock and Dam 22 to Cape Girardeau, MO gage is 3.4 feet on the Mississippi River. The average decrease in stage on the Illinois River is 3.2 feet and on the Missouri River is 1.4 feet. The change in the hydrographs because of this alternative is shown on Plates SL-2 through SL-14 with the suffix of .3. All levees modeled were set to the 25-year level and are displayed in Tables SL-5 through SL-9. The percent of change from the computed base and alternative peak discharge for each gage site is shown in Tables SL-10 through SL-13.

e. Levee Setbacks, Existing Levee Height

Agricultural levees on the Missouri and Mississippi Rivers were setback from their existing levee alignments. The levee setbacks were set at 150% of existing levee floodway, or to provide a minimum floodway width of 5,000 feet, whichever is greater. This alternative examined the effect of levee setbacks on flow conditions with the setback levee height at the existing levee height.

The systemic results for this levee setback levee alternative are displayed in Tables SL-13 through SL-15. The average peak stage decrease from Lock and Dam 22 to Cape Girardeau, MO gage is 1.1

feet on the Mississippi River. The average decrease in stage on the Illinois River is 0.8 feet and average increase in stage on the Missouri River is 0.9 feet. The change in the hydrographs because of this alternative is shown on Plates SL-2 through SL-14 with the suffix of .5. The performance of the levees are displayed in Tables SL-16 through SL-20. The percent of change from the computed base and alternative peak discharge for each gage site is shown in Tables SL-21 through SL-23.

f. Levee Setbacks, No Overtopping

Agricultural levees only on the Mississippi River were setback for this alternative. The setbacks were set at 150% of existing levee floodway, or to provide a minimum floodway width of 5,000 feet, whichever is greater. This alternative assumed that the new setback levees were not overtopped but contained the 1993 flood.

The systemic results for this levee setback levee alternative are displayed in Tables SL-13 through SL-15. The average peak stage deceased slightly with this alternative for the three rivers. The change in the hydrographs because of this alternative is shown on Plates SL-2 through SL-14 with the suffix of .5. The performance of the levees are displayed in Tables SL-16 through SL-20. The percent of change from the computed base and alternative peak discharge for each gage site is shown in Tables SL-21 through SL-23.

10. Other Systemic Alternatives

a. Without Federal Reservoirs

Simulation of this alternative was performed to assess the effect of Federal reservoirs. Discharges were recomputed downstream of all Federal reservoirs assuming that the reservoirs were removed. Within the St. Louis District, the Federal reservoirs that affected 1993 flood consisted of the Mark Twain Lake (R.M. 63.0)on the Salt River, and Lake Shelbyville (R.M. 221.8) and Carlyle Lake (R.M. 106.6) on the Kaskaskia River. The without reservoir discharges were computed at the New London, MO gage (R.M. 35.3) on the Salt River, and Venedy Station (57.2) on the Kaskaskia River.

The systemic results for removing Federal reservoirs are displayed in Tables SL-2 through SL-4. The peak stage increase varies from 0.3 feet at Lock and Dam 22 to 4.1 feet at Lock and Dam 26, and from the Chain of Rocks gage to Cape Girardeau, MO gage average increase is 4.0 feet on the Mississippi River. The average increase in stage on the Illinois River is 2.9 feet and on the Missouri River is 3.9 feet. The change in the hydrographs because of this alternative is shown on Plates SL-2 through SL-14 with the suffix of .4. The levee performance of Mississippi River, Illinois River and Missouri River are displayed in Tables SL-5 through SL-9. The City of St. Louis, East St. Louis and Prairie du Pont urban levees are overtopped in this alternative, as shown in Table SL-6. The percent of change from the computed base and alternative peak discharge for each gage site is shown in Tables SL-10 through SL-13.

b. 5% and 10% Runoff Reduction

For these alternatives, the observed runoff hydrographs from all tributaries to the Missouri and Mississippi Rivers for the 1993 flood were reduced by 5 and 10 percent. The reduction was performed on each ordinate, resulting in a total volume reduction. Large retention structures on all tributaries would

be needed to result in this type of total hydrograph reduction. This alternative will be discussed in detail by St. Paul district.

The systemic results for the runoff reduction of 5 and 10 percent are displayed in Tables SL-2 through SL-4. The average peak stage decrease from Lock and Dam 22 to Lock and Dam 26 is 0.5 and 1.6 feet, and from the Chain of Rocks gage to Cape Girardeau, MO gage is 0.9 and 1.4 feet, respectively on the Mississippi River. The average decrease in stage on the Illinois River is 0.4 and 1.9 feet, and on the Missouri River is 0.5 and 0.4 feet respectively. The change in the storm hydrographs because of this alternative is shown on Plates SL-2 through SL-14 with the suffix of .4. The levee performance of Mississippi River, Illinois River and Missouri River are displayed in Tables SL-5 through SL-9. The percent of change from the computed base and alternative peak discharge for each gage site is shown in Tables SL-10 through SL-13.

11. Additional Alternatives

a. Reservoir Operation

After the 1993 flood, an evaluation of the operation of the St. Louis Districts reservoirs was made. The district has five reservoirs, but two of them were excluded from the study for reasons discussed below. The other three were found to have been operated in a superior manner.

Lake Wappapello on the St. Francis River had no impact on the Mississippi River flooding during 1993. The St. Francis River confluence with the Mississippi is near Memphis, TN far south of the major flooding on the Mississippi River during 1993. A detailed study of Lake Wappapello was not needed and was not conducted.

Rend Lake is on the Big Muddy River. Its confluence with the Mississippi River was within the area of major flooding in 1993. However, Rend Lakes' outflow is though an uncontrolled spillway. The outflow from the lake is determined by the lake level and no reservoir operation is performed.

The Kaskaskia River has two reservoirs that provided flood protection during the flood of 1993. The Kaskaskia Rivers' confluence with the Mississippi River is approximately at Chester, Illinois. This area was impacted by the 1993 flood. Lake Shelbyville and Carlyle Lake operate as a system. Except for backwater from the Mississippi River, the Kaskaskia River experienced no flood damage during the 1993 flood. The discharge from these two reservoirs did **not** add to the many crests or the duration of the 1993 flood. In fact, every crest of the Mississippi River in 1993 was reduced by the operation of these two projects. The operation of these two reservoirs did not prolong the duration of the flood. The two Kaskaskia reservoirs were both success stories during the 1993 flood.

Mark Twain Lake on the Salt River was an exceptionally successful case. Extremely close coordination with the downstream landowners (Lower Salt River Basin Association) played a critical role. Close coordination and frequent special internal river forecasts allowed the water control manager to release water at the optimum time and provide the maximum possible flood control benefits for both the Salt River and Mississippi River basins. The Mark Twain flood control pool was filled and emptied 3.5 times during 1993 with not a single damaging release. The regulation at Mark Twain Lake was superior.

The St. Louis Districts' three reservoir projects all had a positive impact on the 1993 flood. No changes are needed to the water control manuals for these projects based on post-flood analysis.

b. Additional Reservoirs

During the 1960's , five reservoirs for the Meramec River Basin were proposed. The operation of these reservoirs, if constructed, would not have significantly impacted the 1993 flood peak stages because damaging rainfalls generally did not hit the Meramec Basin until well after the 1993 crest. These reservoir projects would have greatly reduced Meramec flooding in September 1993, however.

c. Floodfighting

The Columbia Levee District (R.M. 166.0-156.0) on the Mississippi River is the first agricultural levee downstream of St. Louis. The levee district fought the raising waters for weeks, but the levee was overtopped on the morning of 1 August 1993 and the floodwaters regained their floodplain. The levee design was a 2% annual chance (50-year) flood but because of the valiant efforts of the floodfighters, the levee far exceeded the design. Within an hour after the Columbia Levee District overtopped, a measurable drop in stages was observed at St. Louis, 14 miles upstream of the levee district. The official peak at the St. Louis gage (R.M.179.6) was 49.58 on 1 August occurring about the same time the Columbia Levee District overtopped.

Floodfighting can affect stages both upstream and downstream of the floodfight area. The St. Louis District UNET model was used to simulate a no floodfight scenario at Columbia. The results showed that peak stage could be reduced as much as 1.3 foot at the St. Louis gage, but downstream stages could increase as much as 0.6 foot at Chester gage (109.9), if no floodfighting took place. This levee district protected 13560 acres of farmland and 65 homes. This evaluation indicates that floodfighting could cause additional flooding upstream, but it could also reduce flooding downstream. If floodfighting was not occurring at the Columbia Levee District, the downstream community of Ste Geneieve, MO (R.M. 123.5) may have flooded.

12. Case Studies and Special Studies

a. Chesterfield-Monarch levee-Case Study

The Chesterfield-Monarch earthen levee extends for about 11.5 miles along the Missouri River from river mile 38.5 to 46.0 upstream of the Mississippi River. This privately financed levee protects about 4,240 acres of flood plain lands. About 1,450 acres are currently developed with about 3.1 million square feet of commercial floor space. The levee breached during the 1993 flood. The flood frequency of the 1993 flood was above the 1% chance (100-year) flood in the Chesterfield-Monarch area. The local community is now in the process of recertifying the levee protection to the 1% chance flood, meeting FEMA standards.

Because of additional development in this area, the local community has requested the Corps to study increasing the levee protection to a 0.2% chance (500-year) flood. The expectation is that a 500-year levee would not have broken during the 1993 flood event. This higher Chesterfield-Monarch levee measure was simulated using the UNET model to capture any impacts on the 1993 flood.

The flood elevation impacts of the higher levee were calculated upstream and downstream of the Chesterfield-Monarch levee area by the UNET model. The largest water surface increase just upstream of the higher levee for the 1993 flood was 0.8 foot. Earth City and Riverport levees would not have overtopped if Chesterfield-Monarch levee had held.

b. River Des Peres-Case Study

River Des Peres watershed comprises portions of east-central and south St. Louis County, and west-central and south St. Louis City. The watershed consists of 111 square miles of predominantly urban watershed. River Des Peres enters the Mississippi River at river mile 171.9. The task of this special study is to determine whether a buy-out or levee/floodwall construction would be the less expensive for urban protection.

Flooding on the River Des Peres occurs from two separate sources: Mississippi River backwater and local heavy rainfall. Mississippi River backwater causes flooding on the lower portion of River Des Peres and Gravois Creek when the St. Louis gage is above 36 feet. This report will only address the Mississippi River backwater flooding.

An existing line of temporary levee protection from the Mississippi River backwater was built after the 1973 Mississippi River Flood. The levee protection on the St. Louis City property is to a stage of 45 foot and levee protection in the St. Louis County area is to a stage of 42 foot on the St. Louis gage. To protect to these stages requires extensive pumping from portable and permanent pumping plants to alleviate interior flooding from existing combined sewers, seepage and stormwater.

To achieve urban protection for River Des Peres similar to the City of St. Louis urban protection, a combination of levees, buyouts and floodproofing measures will coincide with interior control measures of pumps, closure gates and pressure sewers. Urban protection from the Mississippi River for River Des Peres would require an elevation of 427 ft NGVD.

c. MR&T Levees-Special Study

The Congressionally authorized flood control project for the lower Mississippi River and Tributaries (MR&T) is designed to contain the "project flood" from Cairo, IL to New Orleans, Louisiana. This MR&T design flood is defined as the greatest flood having a reasonable probability of occurrence, without denoting a specific design frequency. This special study evaluates a similar system from the Cairo, IL to the mouth of the Missouri River.

The design of the "project flood" was reviewed in the 1950's. Some 35 different hypothetical combinations of historical storms were sequentially arranged to conform with frontal movements and synoptic situations consistent with those in nature, to determine the meteorologically feasible pattern that would produce the greatest runoff in the lower Mississippi River. This extensive analysis for the lower Mississippi River was not performed for the middle Mississippi reach (Cairo, Illinois, to the mouth of the Missouri River at St. Louis, Missouri) for this assessment. The design for the middle Mississippi River was accomplished using the established "urban design flood".

The "urban design flood" is defined as a discharge of 1,300,000 cfs at St. Louis, Missouri, adjusted for additional discharge from drainage area downstream of St. Louis, to a discharge of 1,460,000 cfs at

Cairo, Illinois (Mississippi River flow only). At the time the urban levees were designed, this was considered to be the approximate discharge of the 1844 flood. Current frequency studies estimate that this discharge is at least a 0.2% annual chance (500-year) flood. The observed discharge hydrographs of the 1993 flood were adjusted upward to obtain a possible urban design discharge hydrograph and routed with UNET. The resultant elevations represent the height of the levees needed from St. Louis, Missouri, to Cairo, Illinois, to contain the urban design flood". For the Flood Plain Management Assessment analysis, the "urban design flood" for the middle Mississippi was considered to be similar to the "project flood" for the lower Mississippi. The required levee heights were adjusted to account for various hydrologic uncertainties.

The flood elevation impacts of containing this design flood between levees extending from St. Louis, Missouri, to Cairo, Illinois, are significant. For example: a. At the St. Louis Gage (river mile 179.6), the existing urban height flood protection levee would have to be raised about 5 feet; b. For Bois Brule Drainage and Levee District (river mile 95.0-109.5) an agricultural design levee, the average levee height raise would be 11 feet; c. At the Cape Girardeau, Missouri Gage (river mile 52.0), the urban protection levee and floodwall would have to be raised about 5 feet to contain a flood of similar magnitude used for the design of the lower Mississippi River flood protection design.

Raising levees to contain the "urban design flood" within the St. Louis District would result in increased peak flows in the middle Mississippi River and could affect flood stages up to and including the MR&T Project Flood level near and downstream of Cairo, Illinois. The evaluation of these potential impacts are complex and are beyond the scope of this analysis. However, any future studies that consider changes in the present Middle Mississippi levee system should include the evaluation of these downstream effects.

11. References

The Great Flood of 1993 Post-Flood Report, Lower Missouri River Basin, Main Report, U.S. Army Corps of Engineers, North Central Division, September, 1994.

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A Blueprint for Change Part V, Science For Floodplain Management Into The 21st Century, Preliminary Report of the Scientific Assessment and Strategy Team, Report of the Interagency Floodplain Management Review Committee, To the Administration Floodplain Management Task Force, Washington, D.C., June 1994.

TABLE SL-1
COMPARISON OF OBSERVED AND COMPUTED STAGES
1993 FLOOD EVENT

STREAM	GAGE LOCATION	RIVER MILE	GAGE ZERO	1993 OBSERVED STAGE	1993 COMPUTED STAGE
MISSISSIPPI RIVER	LOUISIANA, MO	282.9	437.33	28.4	28.5
MISSISSIPPI RIVER	L&D 24 TW	273.2	421.81	37.7	38.3
MISSISSIPPI RIVER	MOZIER LANDING, IL	260.3	0.00	452.9	452.1
MISSISSIPPI RIVER	L&D 25 TW	241.2	407.09	39.5	40.6
MISSISSIPPI RIVER	GRAFTON, IL	218.3	403.79	38.2	39.3
MISSISSIPPI RIVER	L&D 26 TW	199.9	395.48	42.7	43.1
MISSISSIPPI RIVER	ST. LOUIS, MO	179.6	379.94	49.6	49.8
MISSISSIPPI RIVER	BRICKEYS LANDING, MO	136.0	357.78	47.0	47.3
MISSISSIPPI RIVER	CHESTER, IL	109.9	341.05	49.7	49.9
MISSISSIPPI RIVER	CAPE GIRARDEAU, MO	52.0	304.77	48.5	48.5
ILLINOIS RIVER	MEREDOSIA, IL	70.8	0.00	444.9	445.6
ILLINOIS RIVER	VALLEY CITY, IL	61.3	0.00	444.0	444.7
ILLINOIS RIVER	PEARL, IL	43.2	0.00	442.8	443.2
ILLINOIS RIVER	HARDIN, IL	21.6	0.00	442.3	443.0
MISSOURI RIVER	HERMANN, MO	97.9	481.56	36.2	36.1
MISSOURI RIVER	ST. CHARLES, MO	28.3	413.58	39.6	40.4

TABLE SL-2
PEAK STAGE DIFFERENCE FROM COMPUTED
1993 FLOOD EVENT FOR THE MISSISSIPPI RIVER

MISSISSIPPI RIVER GAGE	RIVER MILE	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
L&D 22 TW	301.1	-5.0	-1.0	3.5	-3.2	0.3	-0.7	-1.5
LOUISIANA	282.9	-3.8	-0.8	3.6	-3.9	0.1	-0.7	-1.6
L&D 24 TW	273.2	-2.7	-0.4	3.7	-3.4	0.4	-0.7	-1.5
MOZIER LANDING	260.3	-1.6	0.7	4.6	-2.6	1.0	-0.4	-1.3
STERLING LANDING	250.8	-1.9	1.0	4.3	-3.5	1.4	-0.2	-1.6
L&D 25 TW	241.2	-1.8	1.1	4.1	-4.0	1.8	-0.3	-1.9
DIXON LANDING	228.3	-1.0	1.2	4.6	-4.2	2.7	-0.3	-1.9
GRAFTON	218.3	-0.9	1.4	5.2	-4.3	3.3	-0.3	-1.8
L&D 26 TW	199.9	-0.5	1.5	6.0	-3.9	4.1	-0.7	-1.7
CHAIN OF ROCKS	190.4	-1.5	0.1	6.5	-4.0	4.2	-0.9	-1.7
ST. LOUIS	179.6	-2.7	0.3	6.3	-3.3	3.2	-1.1	-1.2
WATERS POINT	158.5	-4.7	0.0	8.9	-2.9	3.0	-1.0	-0.8
SELMA	145.8	-4.1	0.1	8.9	-3.0	3.9	-1.2	-1.0
BRICKEYS	136.0	-3.9	0.2	8.3	-3.1	5.1	-0.8	-1.2
LITTLE ROCK LANDING	125.5	-4.8	0.1	7.0	-3.1	5.0	-0.7	-1.4
CHESTER	109.9	-7.2	-1.0	5.8	-3.4	4.0	-1.2	-1.8
BISHOP LANDING	100.8	-6.6	-0.6	5.8	-3.3	4.0	-1.0	-1.7
RED ROCK LANDING	94.1	-6.3	-0.5	5.9	-3.4	4.2	-0.9	-1.7
GRAND TOWER	81.9	-5.1	-0.5	5.2	-3.3	4.0	-0.9	-1.7
MOCCASIN SPRINGS	66.3	-4.2	0.5	5.0	-2.8	3.8	-0.8	-1.5
CAPE GIRARDEAU	52.0	-0.8	1.5	4.1	-2.3	3.2	-0.6	-1.2

TABLE SL-3
PEAK STAGE DIFFERENCE FROM COMPUTED
1993 FLOOD EVENT FOR THE ILLINOIS RIVER

ILLINOIS RIVER GAGE	RIVER MILE	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
MEREDOSIA	70.8	-3.0	-0.1	4.8	-2.3	2.0	-0.5	-2.1
VALLEY CITY	61.3	-2.3	0.5	5.3	-2.8	2.6	-0.4	-2.2
FLORENCE	56.0	-1.8	0.8	5.5	-3.1	2.9	-0.4	-2.0
PEARL	43.2	-1.0	1.5	5.8	-3.7	3.4	-0.3	-1.5
HARDIN	21.6	-0.8	1.5	5.4	-4.2	3.4	-0.3	-1.7

TABLE SL-4
PEAK STAGE DIFFERENCE FROM COMPUTED
1993 FLOOD EVENT FOR THE MISSOURI RIVER

MISSOURI RIVER GAGE	RIVER MILE	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
HERMANN	97.9	1.0	4.6	6.8	-0.8	3.6	-0.2	0.6
NEW HAVEN	81.7	-1.0	2.0	5.2	-2.1	4.0	-0.8	-0.9
WASHINGTON	67.6	-0.9	2.3	6.4	-1.5	4.1	-0.7	-1.1
ST. CHARLES	28.2	-2.5	0.9	2.5	-1.2	3.8	-0.2	-0.3

TABLE SL-5
MISSISSIPPI RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

MISSISSIPPI RIVER AGRICULTURAL LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
SNY MIDDLE AREA	296.0	275.0	NO	REMOVED	REMOVED	NO	YES	NO	NO	NO
SNY LOWER AREA	273.0	266.0	NO	REMOVED	REMOVED	NO	YES	NO	NO	NO
RIVERLAND	293.0	286.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
ELSBERRY MAIN	260.0	249.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
KING'S LAKE	251.0	246.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
KISSINGER PL	253.0	250.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
SANDY CREEK	246.0	245.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
FOLEY	245.0	243.5	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
CAP AU GRIS	243.5	241.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
WINFIELD	241.0	239.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
BREVATOR	239.0	238.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
OLD MONROE	238.2	238.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
COLUMBIA BOTTOM	168.7	165.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
CHOUTEAU ISLAND	195.0	189.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
COLUMBIA	166.0	156.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	NO
HARRISONVILLE	156.3	140.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	NO
FT. CHARTRES	140.0	137.5	YES	REMOVED	REMOVED	NO	YES	YES	YES	NO
STRINGTOWN	137.1	130.4	YES	REMOVED	REMOVED	NO	YES	YES	YES	NO
PRAIRIE DU ROCHER	130.1	118.0	NO	REMOVED	REMOVED	NO	YES	YES	NO	NO

YES: MEANS THE LEVEE WAS OVERTOPPED
NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-5 CONTINUED
MISSISSIPPI RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

MISSISSIPPI RIVER AGRICULTURAL LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
STE. GENEVIEVE #2	119.0	116.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
KASKASKIA ISLAND	115.4	111.6	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
BOIS BRULE	110.6	94.8	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
DEGOGNIA	98.8	84.2	NO	REMOVED	REMOVED	NO	YES	YES	NO	NO
GRAND TOWER	81.8	75.7	NO	REMOVED	REMOVED	NO	YES	YES	NO	NO
PRESTON	75.5	65.5	NO	REMOVED	REMOVED	NO	YES	YES	NO	NO
CLEAR CREEK	65.5	57.3	NO	REMOVED	REMOVED	NO	YES	YES	NO	NO
EAST CAPE	57.3	45.8	NO	REMOVED	REMOVED	NO	YES	YES	NO	NO
LEN SMALL	34.4	32.4	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES

YES: MEANS THE LEVEE WAS OVERTOPPED
NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-6
MISSISSIPPI RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

MISSISSIPPI RIVER URBAN LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
WOOD RIVER	202.5	196.0	NO	NO	NO	NO	NO	YES	NO	NO
EAST ST. LOUIS	195.0	175.4	NO	NO	NO	NO	NO	YES	NO	NO
CITY OF ST. LOUIS	187.3	176.3	NO	NO	NO	NO	NO	YES	NO	NO
PRAIRIE DU PONT	175.4	166.4	NO	NO	NO	NO	NO	YES	NO	NO
CAPE GIRARDEAU	52.6	52.0	NO	NO	NO	NO	NO	NO	NO	NO

YES: MEANS THE LEVEE WAS OVERTOPPED
NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-7
ILLINOIS RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

ILLINOIS RIVER AGRICULTURAL LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
MCGEE CREEK	75.1	67.2	NO	REMOVED	REMOVED	NO	NO	NO	NO	NO
WILLOW CREEK	71.0	69.0	NO	REMOVED	REMOVED	NO	NO	NO	NO	NO
MAUVAISE TERRE	66.6	64.8	NO	REMOVED	REMOVED	NO	NO	NO	NO	NO
VALLEY CITY	66.6	63.1	NO	REMOVED	REMOVED	NO	NO	YES	NO	NO
SCOTT COUNTY	63.1	56.7	NO	REMOVED	REMOVED	NO	NO	YES	NO	NO
BIG SWAN	56.6	50.1	NO	REMOVED	REMOVED	NO	NO	YES	NO	NO
HILLVIEW	50.0	43.2	YES	REMOVED	REMOVED	NO	NO	YES	YES	YES
HARTWELL	43.1	38.2	YES	REMOVED	REMOVED	NO	NO	YES	YES	NO
KEACH	38.0	32.8	NO	REMOVED	REMOVED	NO	YES	YES	NO	NO
ELDRED AND SPANKY	32.4	23.8	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
NUTWOOD	23.8	15.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES

YES: MEANS THE LEVEE WAS OVERTOPPED
NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-8
MISSOURI RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

MISSOURI RIVER AGRICULTURAL LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
TRI-COUNTY	96.7	91.8	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
BERGER	94.0	82.4	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
PINCKEY BOTTOM	82.5	72.4	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
HOLTMIEIR	74.4	72.6	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
ST. JOHNS	72.6	69.7	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
DUTZOW	68.2	66.1	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
AUGUSTA BOTTOM	67.5	60.6	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
LABADIE BOTTOM	61.0	54.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
ST. ALBANS	55.5	53.5	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
DARST BOTTOM	56.0	49.3	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
UNIVERSITY OF MO	49.2	47.8	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
BOEHOMME ISLAND	41.7	38.8	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
GREENS BOTTOM	38.8	33.6	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
HOWARD BEND	36.9	30.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
CONSOL NL DIST 1	23.8	15.3	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
CONSOL NL DIST 2	15.3	13.4	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
CONSOL NL DIST 3	13.4	2.5	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
MITTLER & ETC.	21.6	7.4	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
CORA ISLAND	6.3	2.8	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
COLUMBIA BOTTOM	5.3	0.0	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES
KUHS	2.6	0.6	YES	REMOVED	REMOVED	NO	YES	YES	YES	YES

YES: MEANS THE LEVEE WAS OVERTOPPED NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-9
MISSOURI RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

MISSOURI RIVER URBAN LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
MONARCH	44.8	38.3	YES	NO	NO	NO	NO	YES	YES	YES
RIVERPORT	30.7	29.6	NO	NO	NO	NO	NO	NO	NO	NO
EARTH CITY	29.5	27.1	NO	NO	NO	NO	NO	NO	NO	NO

YES: MEANS THE LEVEE WAS OVERTOPPED
NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-10

PERCENT CHANGE FROM COMPUTED BASE DISCHARGE

1993 FLOOD EVENT FOR THE MISSISSIPPI RIVER

MISSISSIPPI RIVER GAGE	RIVER MILE	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
L&D 22 TW	301.1	+ 2	- 4	+34	-24	+ 3	- 8	-14
LOUISIANA	282.9	+ 2	- 4	+34	-28	+ 1	- 6	-13
L&D 24 TW	273.2	+ 3	- 3	+34	-28	+ 1	- 6	-13
MOZIER LANDING	260.3	+ 3	- 3	+34	-28	+ 2	- 5	-12
STERLING LANDING	250.8	+16	+ 9	+50	-18	+ 4	- 5	- 8
L&D 25 TW	241.2	+ 4	- 3	+34	-27	+ 2	- 5	-12
DIXON LANDING	228.3	+ 4	- 4	+33	-27	+ 1	- 5	-13
GRAFTON	218.3	- 3	- 3	+22	-24	- 2	- 8	-13
L&D 26 TW	199.9	- 4	0	+ 7	-24	+ 3	0	- 9
CHAIN OF ROCKS	190.4	+ 2	- 3	+20	-16	+20	0	- 8
ST. LOUIS	179.6	+ 1	- 4	+19	-16	+20	0	- 8
WATERS POINT	158.5	+ 6	+ 1	+24	-11	+ 3	- 4	- 3
SELMA	145.8	+ 7	+ 1	+23	-11	+ 3	- 4	- 3
BRICKEYS	136.0	+ 7	+ 1	+23	-11	+ 3	- 4	- 3
LITTLE ROCK LANDING	125.5	+ 6	+ 1	+22	-12	+15	+ 2	- 2
CHESTER	109.9	+ 8	+ 2	+23	-12	+14	- 4	- 7
BISHOP LANDING	100.8	+ 8	+ 2	+23	-12	+14	- 4	- 7
RED ROCK LANDING	94.1	+ 9	+ 3	+25	-11	+13	- 3	- 6
GRAND TOWER	81.9	+ 9	+ 3	+25	-12	+20	- 3	- 6
MOCCASIN SPRINGS	66.3	+ 9	+ 2	+24	-12	+16	- 3	- 6
CAPE GIRARDEAU	52.0	+ 9	+ 2	+24	-12	+16	- 4	- 6

TABLE SL-11
PERCENT CHANGE FROM COMPUTED BASE DISCHARGE
1993 FLOOD EVENT FOR THE ILLINOIS RIVER

ILLINOIS RIVER GAGE	RIVER MILE	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
MEREDOSIA	70.8	0	0	0	0	0	- 5	-10
VALLEY CITY	61.3	- 2	- 1	- 3	0	- 5	- 6	-11
FLORENCE	56.0	- 4	- 2	- 4	0	- 8	- 6	-11
PEARL	43.2	- 3	- 2	- 4	+ 1	- 9	- 8	-12
HARDIN	21.6	+18	+12	- 5	-10	+10	- 8	-15

TABLE SL-12
PERCENT CHANGE FROM COMPUTED BASE DISCHARGE
1993 FLOOD EVENT FOR THE MISSOURI RIVER

MISSOURI RIVER GAGE	RIVER MILE	LEVEE REMOVED AGRICULTURAL	LEVEE REMOVED NATURAL	CONTAINED	25-YEAR LEVEE	NO RESERVOIR	REDUCE RUNOFF BY 5%	REDUCE RUNOFF BY 10%
HERMANN	97.9	+ 3	- 5	+14	-12	+31	- 5	- 6
NEW HAVEN	81.7	+ 3	- 5	+14	-14	+31	- 6	- 6
WASHINGTON	67.6	+ 2	- 5	+14	-16	+31	- 5	- 7
ST. CHARLES	28.2	+ 3	- 5	+14	-14	+32	- 4	- 8

TABLE SL-13
PEAK STAGE DIFFERENCE FROM COMPUTED
1993 FLOOD EVENT FOR THE MISSISSIPPI RIVER

MISSISSIPPI RIVER GAGE	RIVER MILE	ALL AGRICULTURAL MISSISSIPPI RIVER AND MISSOURI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
L&D 22 TW	301.1	-0.6	2.2
LOUISIANA	282.9	-0.2	2.0
L&D 24 TW	273.2	-1.5	0.4
MOZIER LANDING	260.3	-1.3	0.4
STERLING LANDING	250.8	-1.3	-0.7
L&D 25 TW	241.2	-1.3	-1.3
DIXON LANDING	228.3	-1.1	-0.9
GRAFTON	218.3	-0.8	-0.5
L&D 26 TW	199.9	-1.0	-0.6
CHAIN OF ROCKS	190.4	-1.3	-0.7
ST. LOUIS	179.6	-1.8	-0.5
WATERS POINT	158.5	-2.3	0.1
SELMA	145.8	-2.6	-0.3
BRICKEYS	136.0	-1.5	-0.2
LITTLE ROCK LANDING	125.5	-1.3	-0.4
CHESTER	109.9	-1.1	-0.3
BISHOP LANDING	100.8	-0.7	0.2
RED ROCK LANDING	94.1	-0.4	0.5
GRAND TOWER	81.9	-0.3	0.6
MOCCASIN SPRINGS	66.3	-0.2	0.6
CAPE GIRARDEAU	52.0	-0.2	0.4

TABLE SL-14
PEAK STAGE DIFFERENCE FROM COMPUTED
1993 FLOOD EVENT FOR THE ILLINOIS RIVER

ILLINOIS RIVER GAGE	RIVER MILE	ALL AGRICULTURAL MISSISSIPPI RIVER AND MISSOURI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
MEREDOSIA	70.8	-0.8	-0.5
VALLEY CITY	61.3	-0.9	-0.3
FLORENCE	56.0	-0.8	-0.2
PEARL	43.2	-0.6	-0.1
HARDIN	21.6	-0.7	-0.4

TABLE SL-15
PEAK STAGE DIFFERENCE FROM COMPUTED
1993 FLOOD EVENT FOR THE MISSOURI RIVER

MISSOURI RIVER GAGE	RIVER MILE	ALL AGRICULTURAL MISSISSIPPI RIVER AND MISSOURI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
HERMANN	97.9	0.6	0.0
NEW HAVEN	81.7	1.8	0.0
WASHINGTON	67.6	1.9	0.0
ST. CHARLES	28.2	-0.6	-0.2

TABLE SL-16
MISSISSIPPI RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

MISSISSIPPI RIVER AGRICULTURAL LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	ALL AGRICULTURAL MISSISSIPPI RIVER AND MISSOURI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
SNY MIDDLE AREA	296.0	275.0	NO	NO	NO
SNY LOWER AREA	273.0	266.0	NO	NO	NO
RIVERLAND	293.0	286.0	YES	YES	NO
ELSBERRY MAIN	260.0	249.0	YES	YES	NO
KING'S LAKE	251.0	246.0	YES	YES	NO
KISSINGER PL	253.0	250.0	YES	YES	NO
SANDY CREEK	246.0	245.0	YES	YES	NO
FOLEY	245.0	243.5	YES	YES	NO
CAP AU GRIS	243.5	241.0	YES	YES	NO
WINFIELD	241.0	239.0	YES	YES	NO
BREVATOR	239.0	238.0	YES	YES	NO
OLD MONROE	238.2	238.0	YES	YES	NO
COLUMBIA BOTTOM	168.7	165.0	YES	YES	NO
CHOUTEAU ISLAND	195.0	189.0	YES	YES	NO
COLUMBIA	166.0	156.0	YES	YES	NO
HARRISONVILLE	156.3	140.0	YES	YES	NO
FT. CHARTRES	140.0	137.5	YES	YES	NO
STRINGTOWN	137.1	130.4	YES	YES	NO
PRAIRIE DU ROCHER	130.1	118.0	NO	NO	NO

YES: MEANS THE LEVEE WAS OVERTOPPED
NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-16 CONTINUED
MISSISSIPPI RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

MISSISSIPPI RIVER AGRICULTURAL LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	ALL AGRICULTURAL MISSISSIPPI RIVER AND MISSOURI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
STE. GENEVIEVE #2	119.0	116.0	YES	YES	NO
KASKASKIA ISLAND	115.4	111.6	YES	YES	NO
BOIS BRULE	110.6	94.8	YES	NO	NO
DEGOGNIA	98.8	84.2	NO	NO	NO
GRAND TOWER	81.8	75.7	NO	NO	NO
PRESTON	75.5	65.5	NO	NO	NO
CLEAR CREEK	65.5	57.3	NO	NO	NO
EAST CAPE	57.3	45.8	NO	NO	NO
LEN SMALL	34.4	32.4	YES	YES	NO

YES: MEANS THE LEVEE WAS OVERTOPPED
NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-17
MISSISSIPPI RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

MISSISSIPPI RIVER URBAN LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	ALL AGRICULTURAL MISSISSIPPI RIVER AND MISSOURI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
WOOD RIVER	202.5	196.0	NO	NO	NO
EAST ST. LOUIS	195.0	175.4	NO	NO	NO
CITY OF ST. LOUIS	187.3	176.3	NO	NO	NO
PRAIRIE DU PONT	175.4	166.4	NO	NO	NO
CAPE GIRARDEAU	52.6	52.0	NO	NO	NO

YES: MEANS THE LEVEE WAS OVERTOPPED
NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-18
ILLINOIS RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

ILLINOIS RIVER AGRICULTURAL LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	ALL AGRICULTURAL MISSISSIPPI RIVER AND MISSOURI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
MCGEE CREEK	75.1	67.2	NO	NO	NO
WILLOW CREEK	71.0	69.0	NO	NO	NO
MAUVAISE TERRE	66.6	64.8	NO	NO	NO
VALLEY CITY	66.6	63.1	NO	NO	NO
SCOTT COUNTY	63.1	56.7	NO	NO	NO
BIG SWAN	56.6	50.1	NO	NO	NO
HILLVIEW	50.0	43.2	YES	NO	YES
HARTWELL	43.1	38.2	YES	YES	YES
KEACH	38.0	32.8	NO	NO	NO
ELDRED AND SPANKY	32.4	23.8	YES	YES	YES
NUTWOOD	23.8	15.0	YES	YES	YES

YES: MEANS THE LEVEE WAS OVERTOPPED
NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-19
MISSOURI RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

MISSOURI RIVER AGRICULTURAL LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
TRI-COUNTY	96.7	91.8	YES	YES	YES
BERGER	94.0	82.4	YES	YES	YES
PINCKEY BOTTOM	82.5	72.4	YES	YES	YES
HOLTMIEIR	74.4	72.6	YES	YES	YES
ST. JOHNS	72.6	69.7	YES	YES	YES
DUTZOW	68.2	66.1	YES	YES	YES
AUGUSTA BOTTOM	67.5	60.6	YES	YES	YES
LABADIE BOTTOM	61.0	54.0	YES	YES	YES
ST. ALBANS	55.5	53.5	YES	YES	YES
DARST BOTTOM	56.0	49.3	YES	YES	YES
UNIVERSITY OF MO	49.2	47.8	YES	YES	YES
BOEHOMME ISLAND	41.7	38.8	YES	YES	YES
GREENS BOTTOM	38.8	33.6	YES	YES	YES
HOWARD BEND	36.9	30.0	YES	YES	YES
CONSOL NL DIST 1	23.8	15.3	YES	YES	YES
CONSOL NL DIST 2	15.3	13.4	YES	YES	YES
CONSOL NL DIST 3	13.4	2.5	YES	YES	YES
MITTLER & ETC.	21.6	7.4	YES	YES	YES
CORA ISLAND	6.3	2.8	YES	YES	YES
COLUMBIA BOTTOM	5.3	0.0	YES	YES	YES
KUHS	2.6	0.6	YES	YES	YES

YES: MEANS THE LEVEE WAS OVERTOPPED
NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-20
MISSOURI RIVER LEVEE PERFORMANCE
1993 FLOOD EVENT

MISSOURI RIVER URBAN LEVEE	UPPER RIVER MILE	LOWER RIVER MILE	1993 FLOOD	ALL AGRICULTURAL MISSISSIPPI RIVER AND MISSOURI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
MONARCH	44.8	38.3	YES	YES	YES
RIVERPORT	30.7	29.6	NO	NO	NO
EARTH CITY	29.5	27.1	NO	NO	NO

YES: MEANS THE LEVEE WAS OVERTOPPED
NO: MEANS THE LEVEE WAS NOT OVERTOPPED

TABLE SL-21
PERCENT CHANGE FROM COMPUTED BASE DISCHARGE
1993 FLOOD EVENT FOR THE MISSISSIPPI RIVER

MISSISSIPPI RIVER GAGE	RIVER MILE	ALL AGRICULTURAL MISSISSIPPI RIVER AND MISSOURI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
L&D 22 TW	301.1	-17	- 2
LOUISIANA	282.9	-15	- 2
L&D 24 TW	273.2	-14	- 1
MOZIER LANDING	260.3	-14	- 1
STERLING LANDING	250.8	-12	+12
L&D 25 TW	241.2	-13	+ 1
DIXON LANDING	228.3	-12	+ 1
GRAFTON	218.3	-10	- 4
L&D 26 TW	199.9	- 2	- 2
CHAIN OF ROCKS	190.4	- 1	- 2
ST. LOUIS	179.6	- 1	- 3
WATERS POINT	158.5	- 5	+ 2
SELMA	145.8	- 5	+ 2
BRICKEYS	136.0	- 6	+ 2
LITTLE ROCK LANDING	125.5	+ 1	+ 2
CHESTER	109.9	0	+ 3
BISHOP LANDING	100.8	0	+ 3
RED ROCK LANDING	94.1	+ 1	+ 5
GRAND TOWER	81.9	+ 1	+ 5
MOCCASIN SPRINGS	66.3	+ 1	+ 5
CAPE GIRARDEAU	52.0	+ 1	+ 5

TABLE SL-22
PERCENT CHANGE FROM COMPUTED BASE DISCHARGE
1993 FLOOD EVENT FOR THE ILLINOIS RIVER

ILLINOIS RIVER GAGE	RIVER MILE	ALL AGRICULTURAL MISSISSIPPI RIVER AND MISSOURI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
MEREDOSIA	70.8	0	0
VALLEY CITY	61.3	0	- 1
FLORENCE	56.0	0	- 2
PEARL	43.2	+ 2	- 3
HARDIN	21.6	- 2	- 8

TABLE SL-23
PERCENT CHANGE FROM COMPUTED BASE DISCHARGE
1993 FLOOD EVENT FOR THE MISSOURI RIVER

MISSOURI RIVER GAGE	RIVER MILE	ALL AGRICULTURAL MISSISSIPPI RIVER AND MISSOURI RIVER LEVEES-SETBACK AT EXISTING HEIGHT	ALL AGRICULTURAL MISSISSIPPI RIVER LEVEES-SETBACK AT CONTAINED HEIGHT
HERMANN	97.9	0	0
NEW HAVEN	81.7	0	0
WASHINGTON	67.6	0	0
ST. CHARLES	28.2	0	0

Stream components in the Mississippi River UNET model.

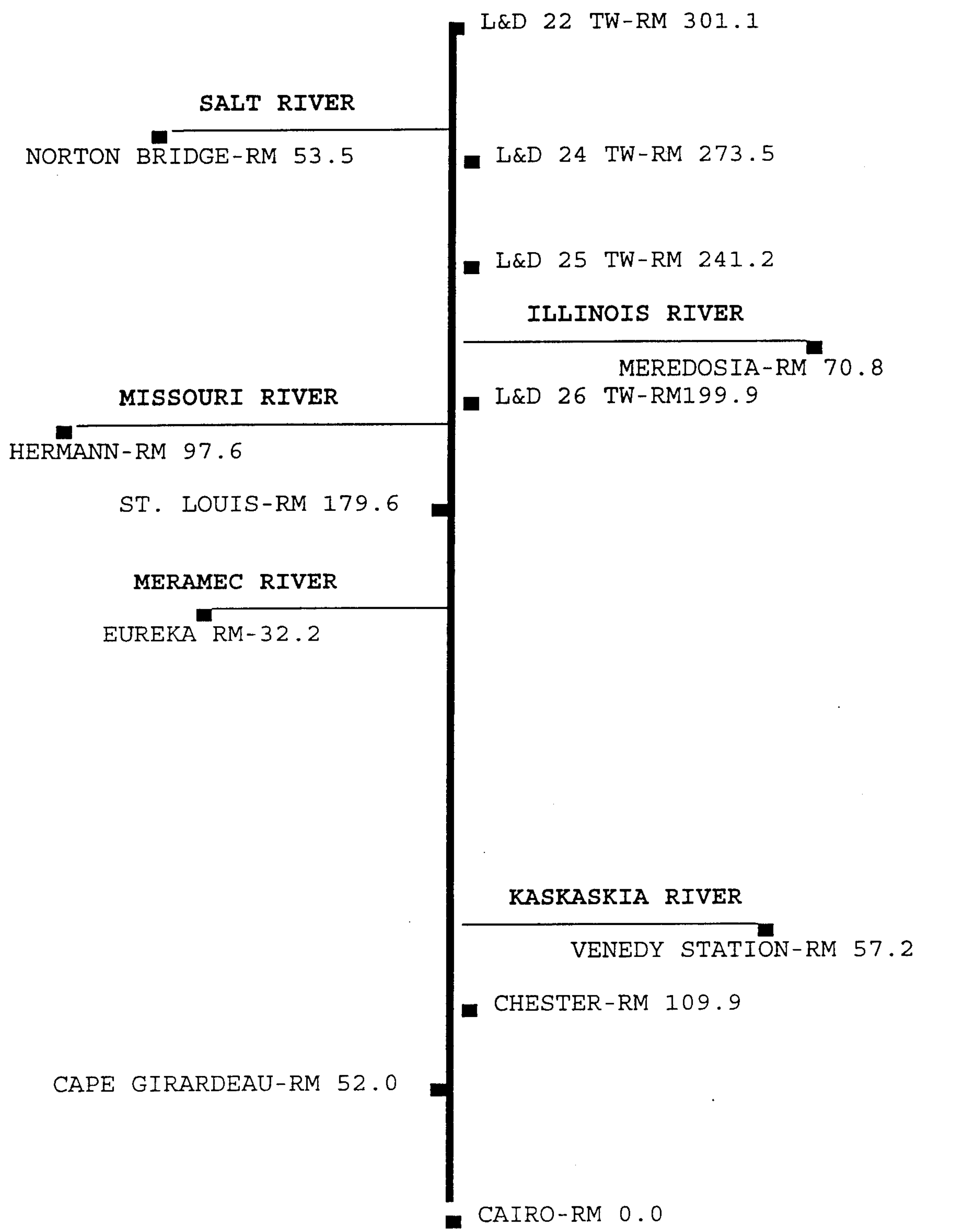


PLATE SL-1

Stream components in the Mississippi River UNET model.

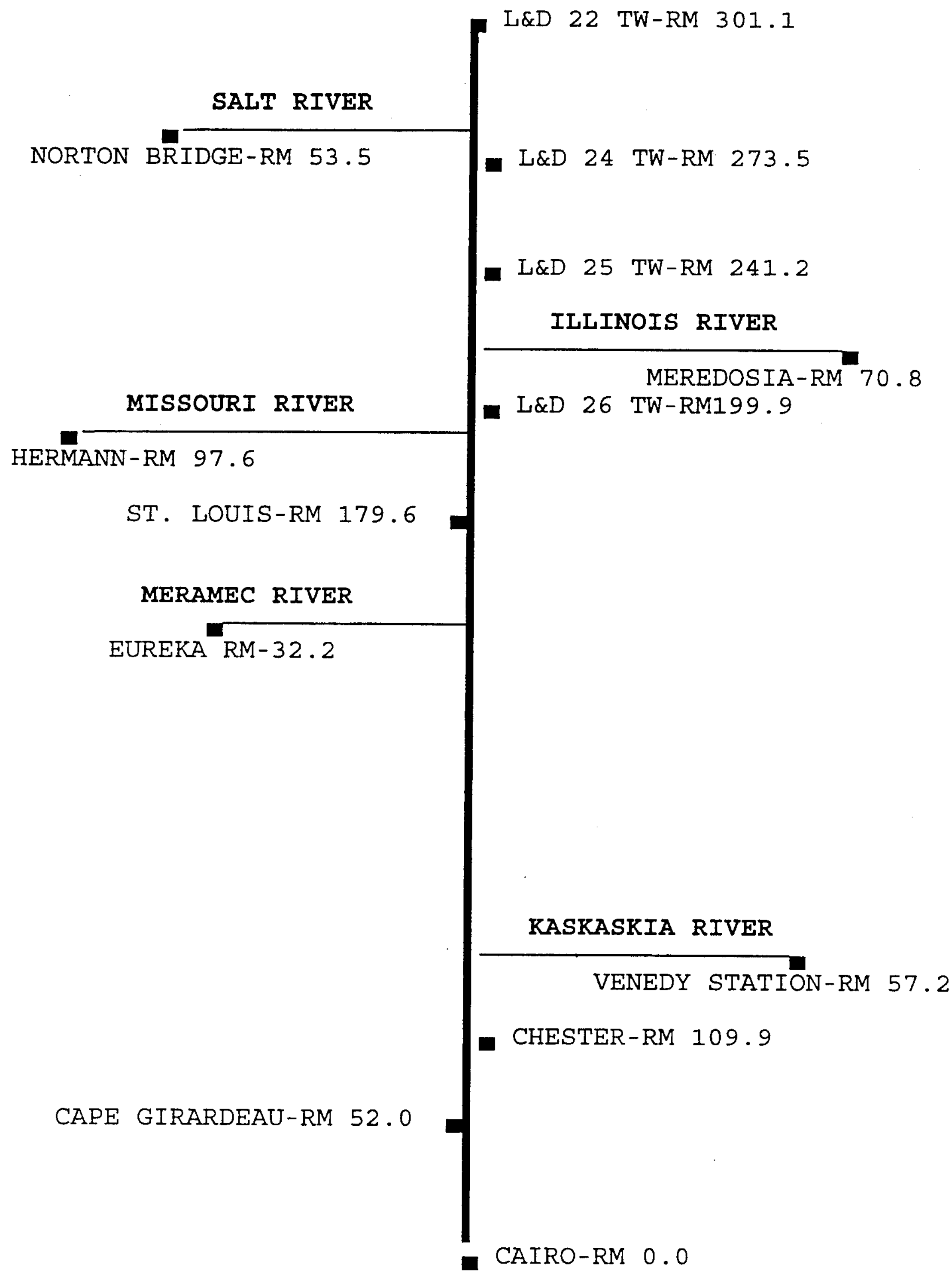


PLATE SL-1

MISSISSIPPI RIVER
L& D 22 TW GAGE - RM 301.1
COMPUTED VS OBSERVED STAGES -1993 FLOOD

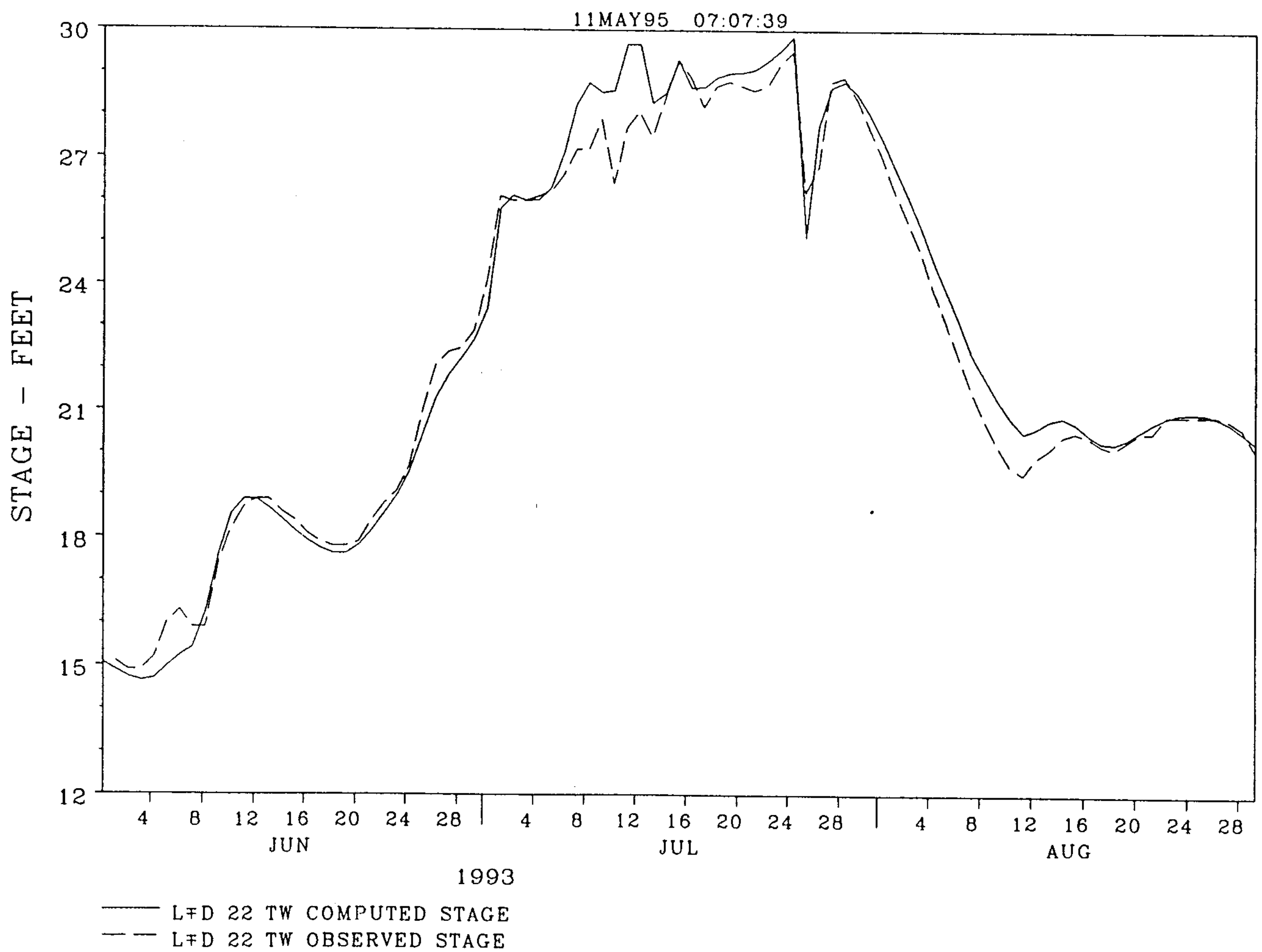


PLATE SL-2.1

MISSISSIPPI RIVER
L &D 22 TW GAGE - RM 301.1
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

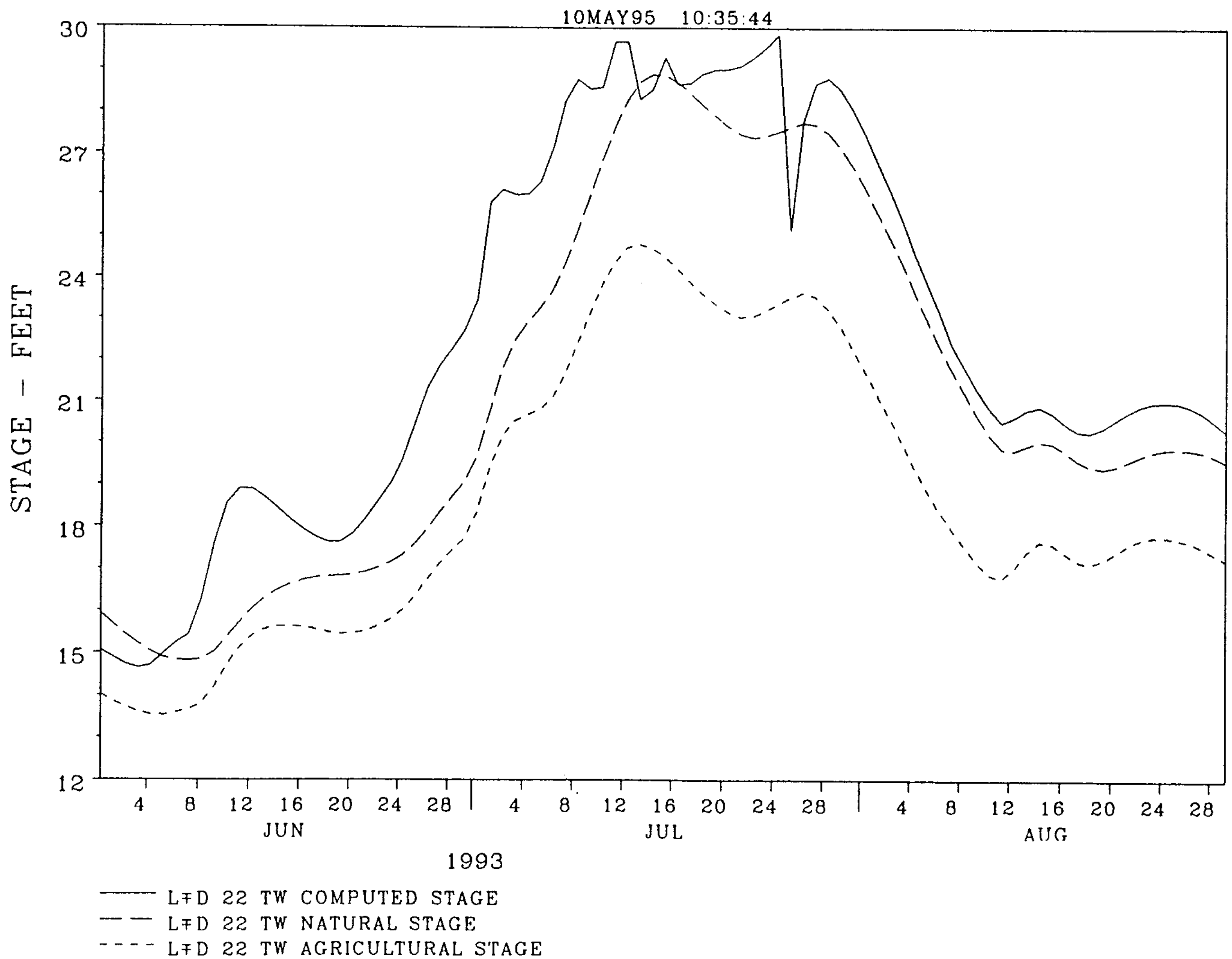
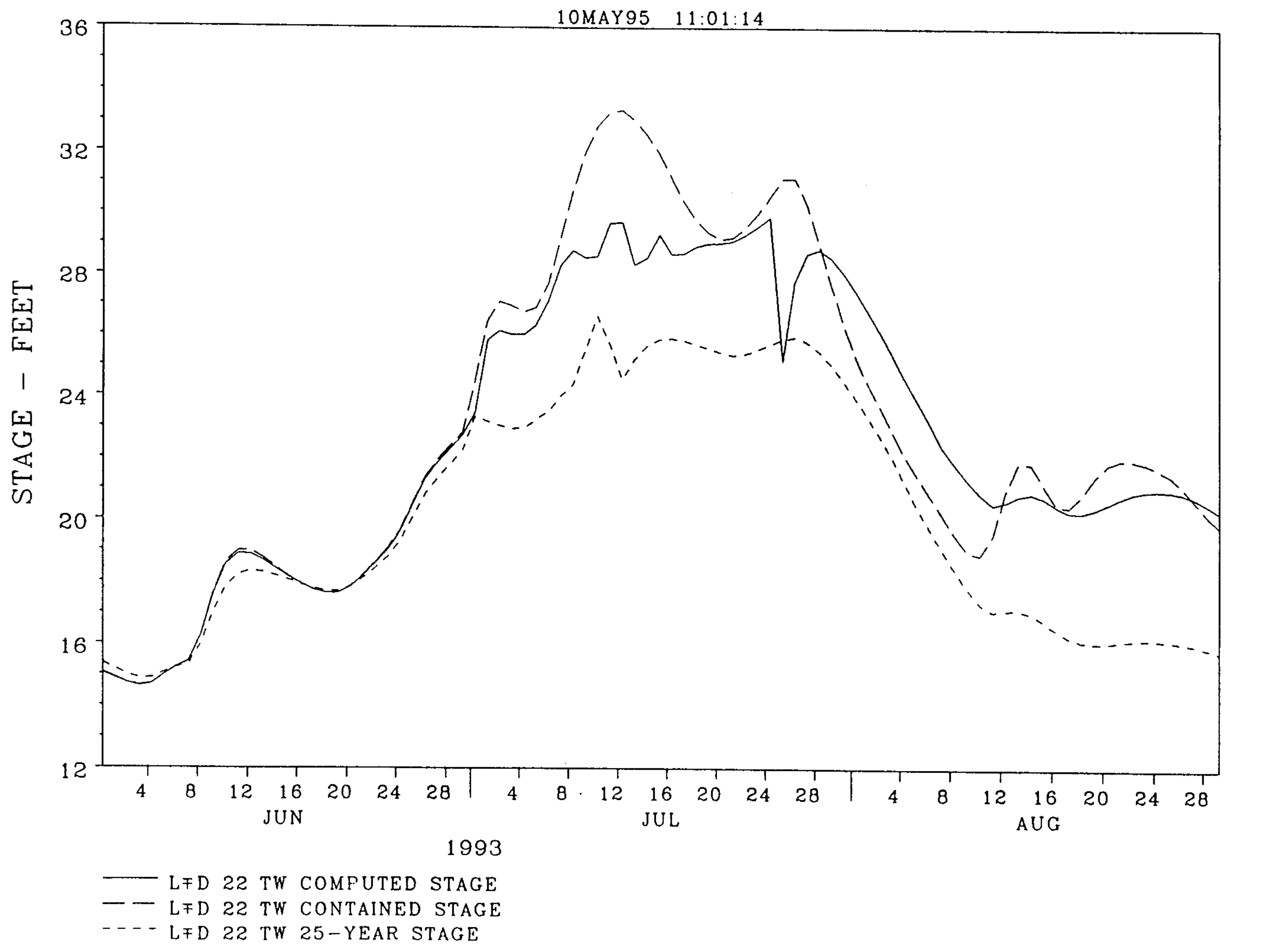
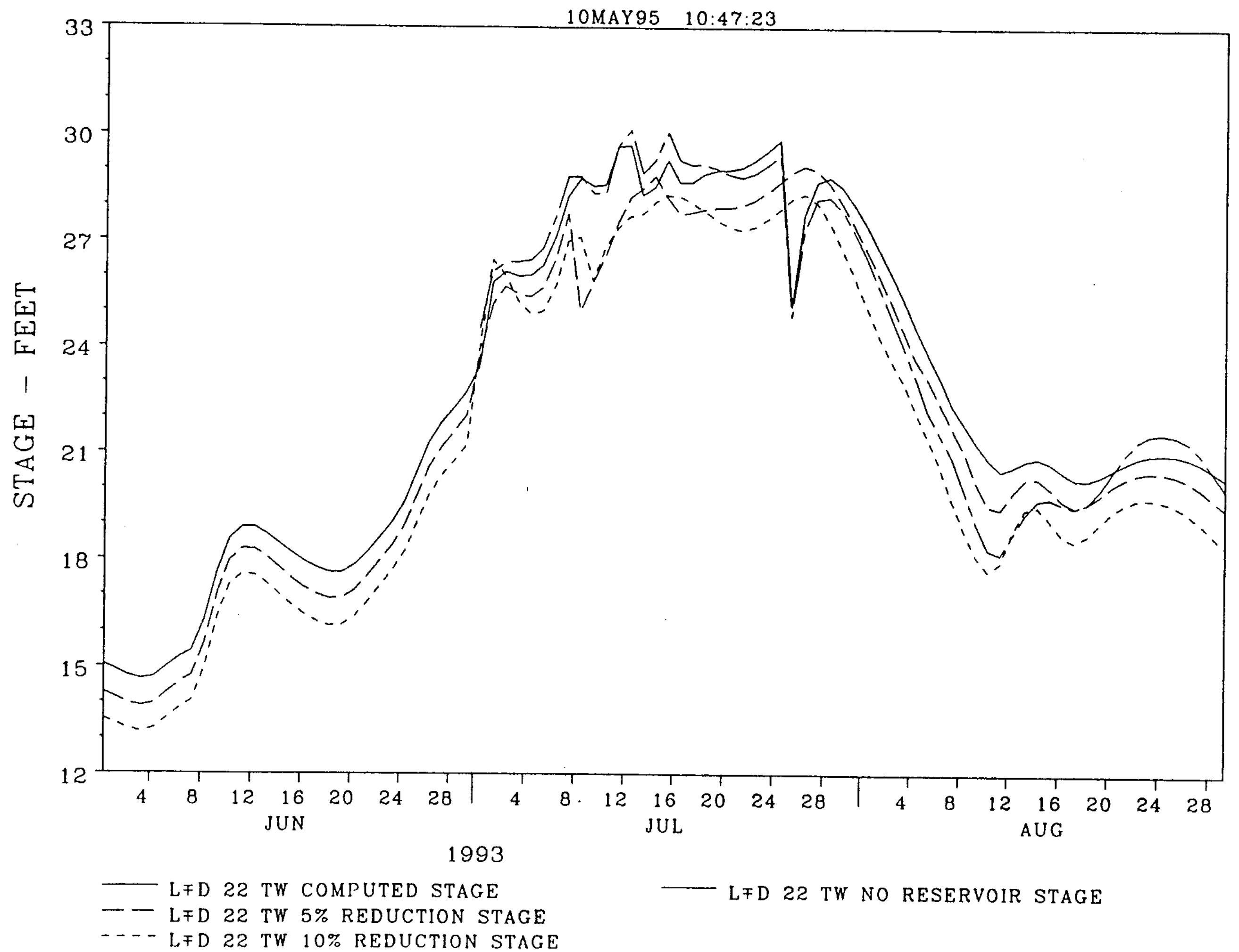


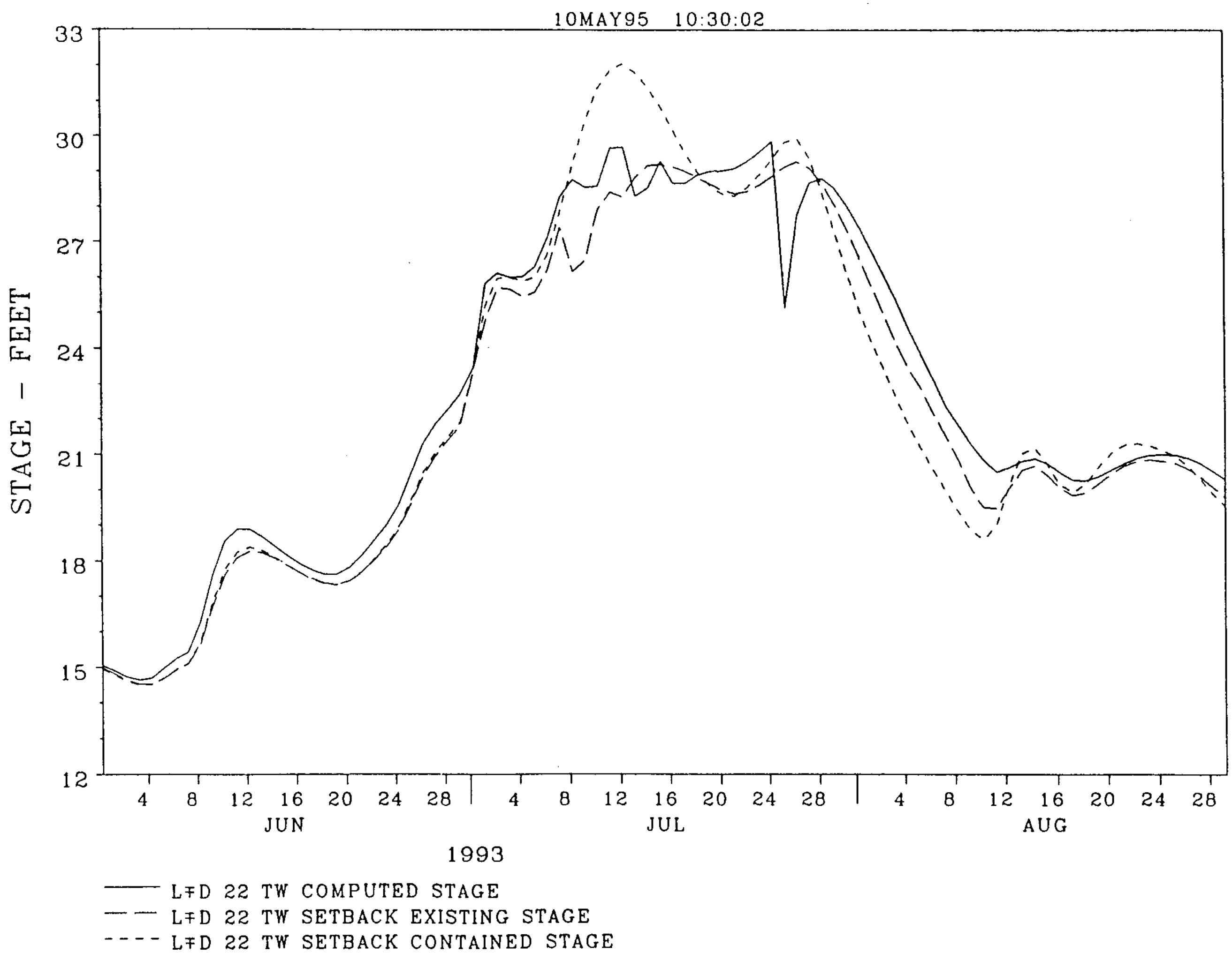
PLATE SL-2.2


MISSISSIPPI RIVER
L &D 22 TW GAGE - RM 301.1
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES
10MAY95 11:01:14
STAGE - FEET
36
32
28
24
20
16
12
4 8 12 16 20 24 28
JUN
4 8 12 16 20 24 28
JUL
4 8 12 16 20 24 28
AUG
1993
L+D 22 TW COMPUTED STAGE
L+D 22 TW CONTAINED STAGE
L+D 22 TW 25-YEAR STAGE

MISSISSIPPI RIVER
L&D 22 TW GAGE – RM 301.1
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS



MISSISSIPPI RIVER
L&D 22 TW GAGE - RM 301.1
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS



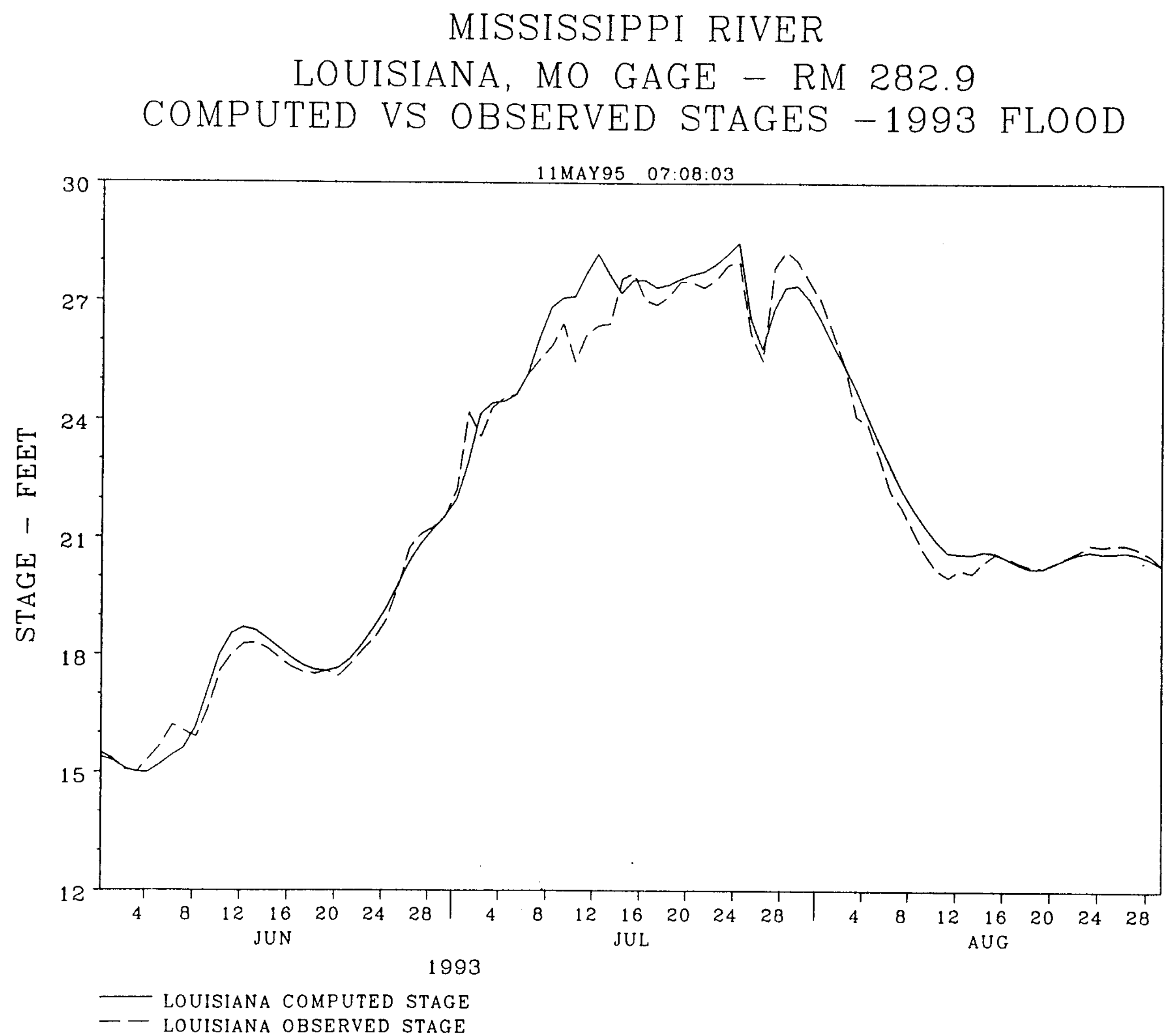

MISSISSIPPI RIVER
LOUISIANA, MO GAGE - RM 282.9
COMPUTED VS OBSERVED STAGES -1993 FLOOD
11MAY95 07:08:03
STAGE - FEET
30
27
24
21
18
15
12
4 8 12 16 20 24 28
JUN
4 8 12 16 20 24 28
JUL
4 8 12 16 20 24 28
AUG
1993
LOUISIANA COMPUTED STAGE
LOUISIANA OBSERVED STAGE

PLATE SL-3.1

MISSISSIPPI RIVER
LOUISIANA, MO GAGE – RM 282.9
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

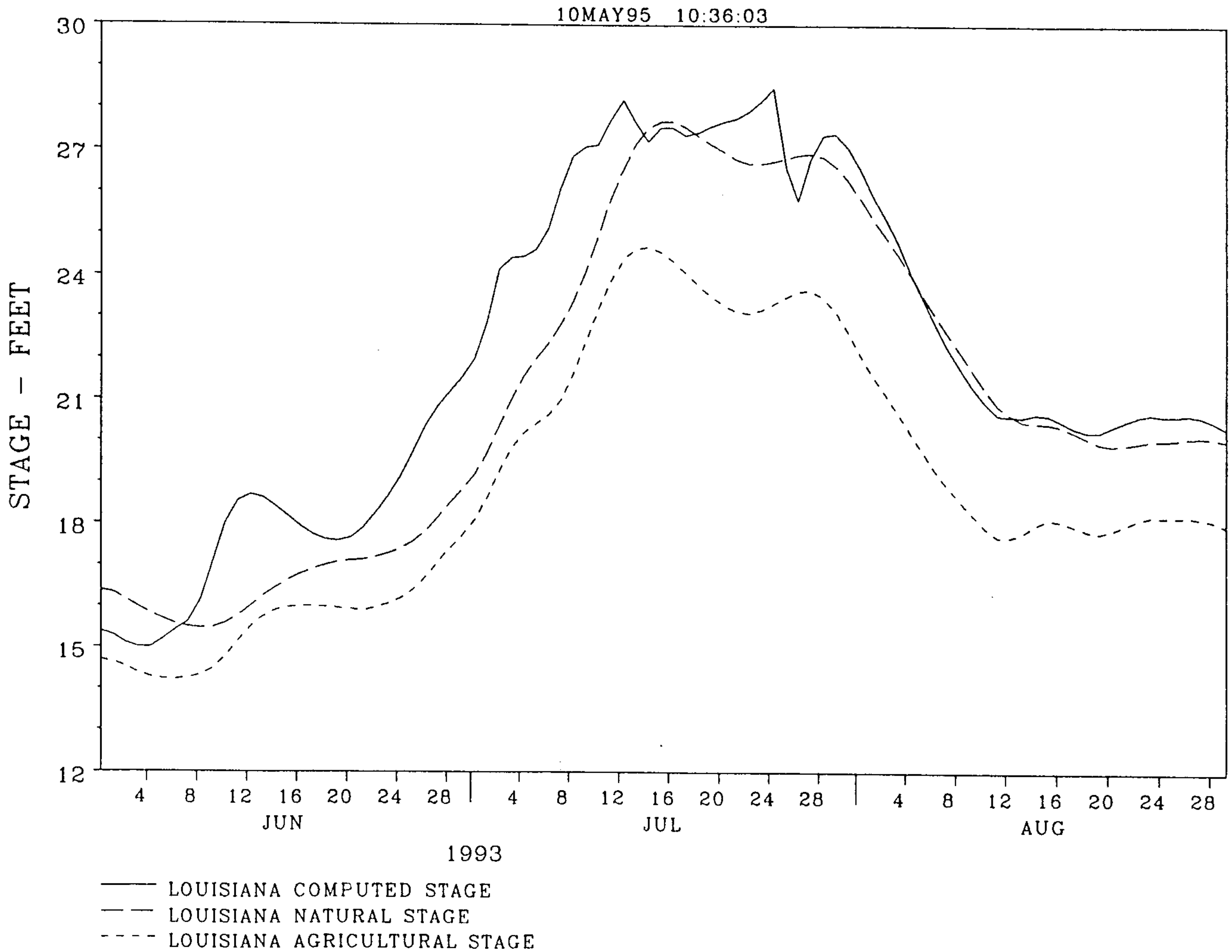
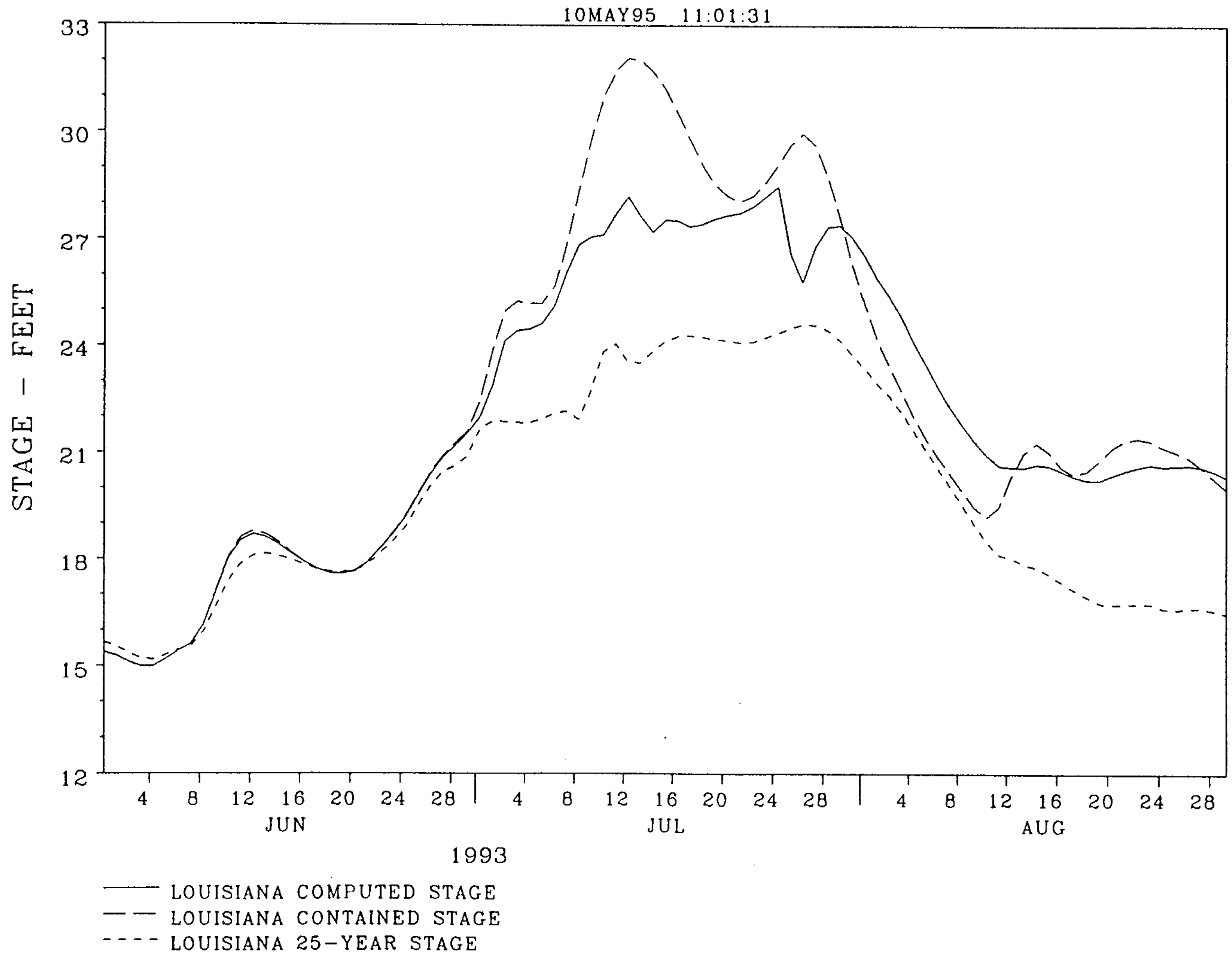


PLATE SL-3.2

MISSISSIPPI RIVER
LOUISIANA, MO GAGE – RM 282.9
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



MISSISSIPPI RIVER
LOUISIANA, MO GAGE – RM 282.9
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS

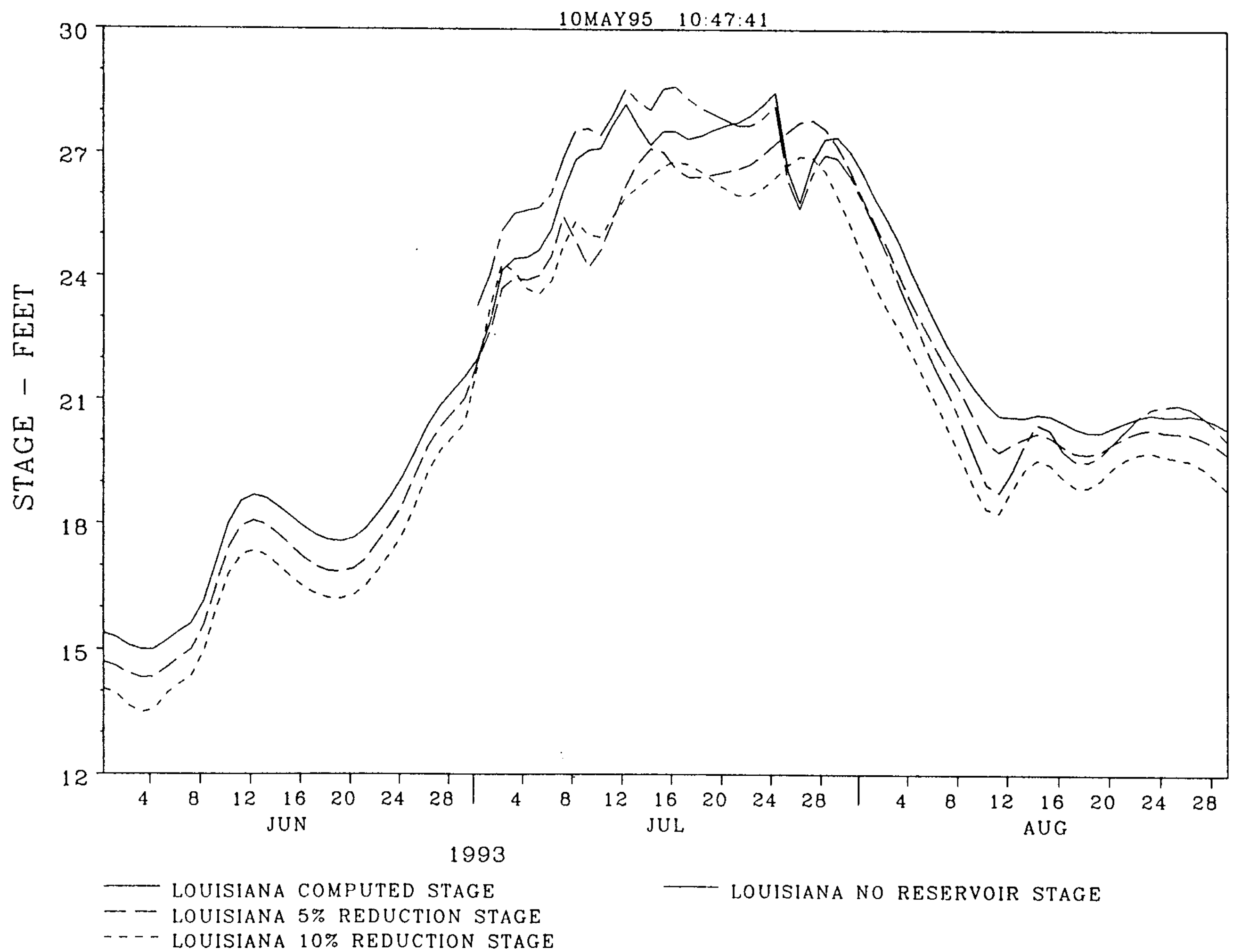


PLATE SL-3.4

MISSISSIPPI RIVER
LOUISIANA, MO GAGE – RM 282.9
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS

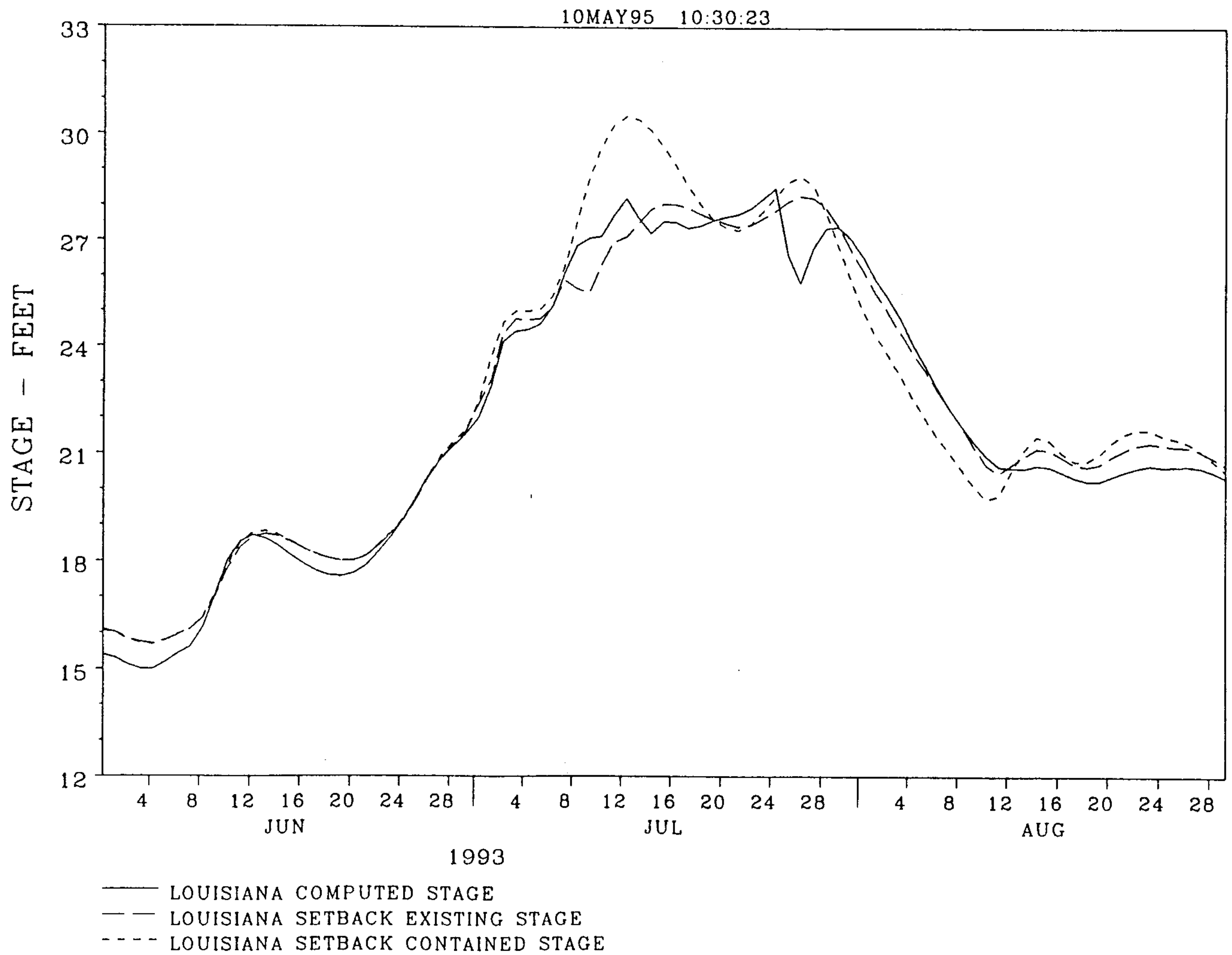


PLATE SL-3.5

MISSISSIPPI RIVER
L&D 24 TW GAGE - RM 273.5
COMPUTED VS OBSERVED STAGES -1993 FLOOD

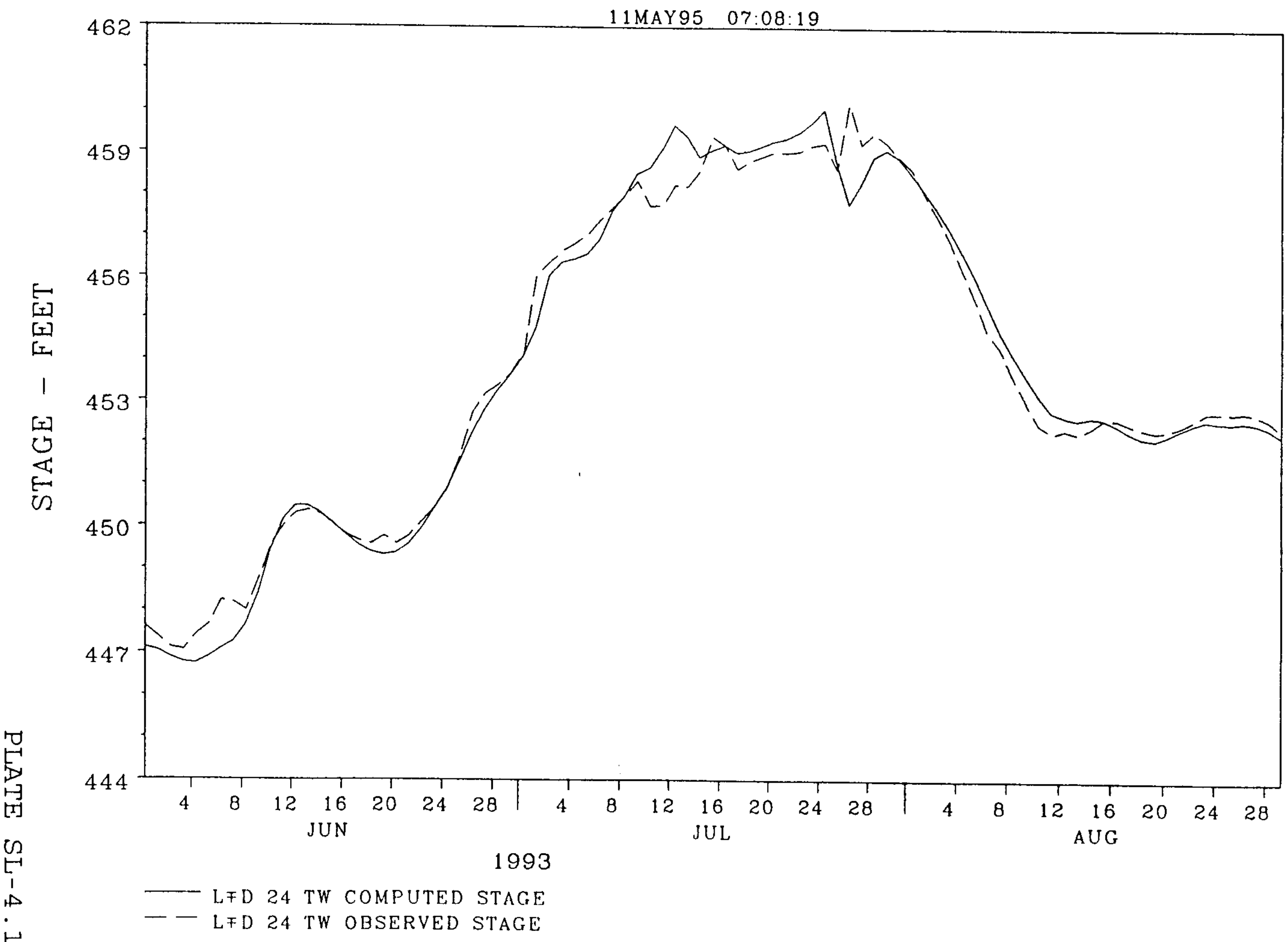


PLATE SL-4.1

MISSISSIPPI RIVER
L&D 24 TW GAGE - RM 273.5
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

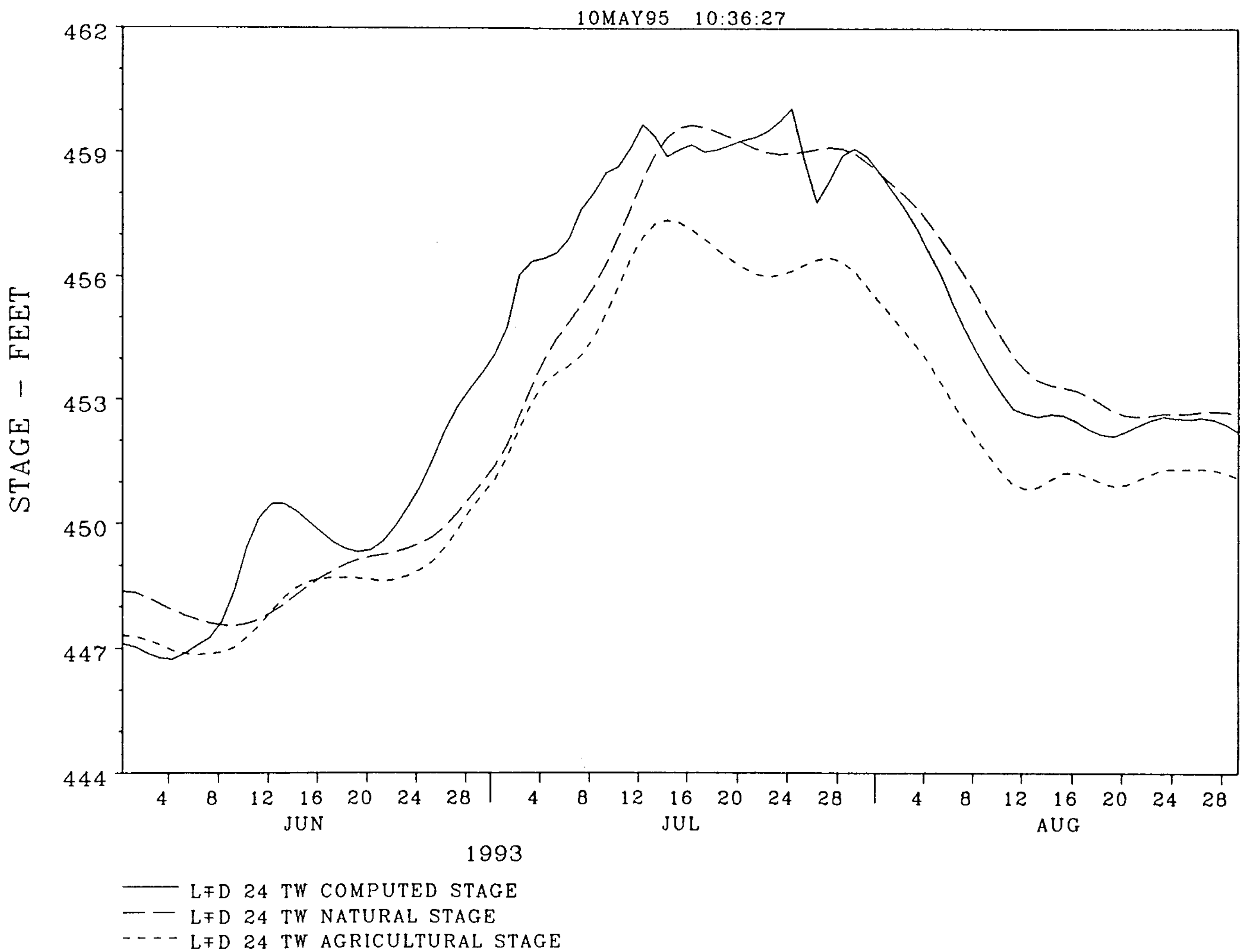


PLATE SL-4.2

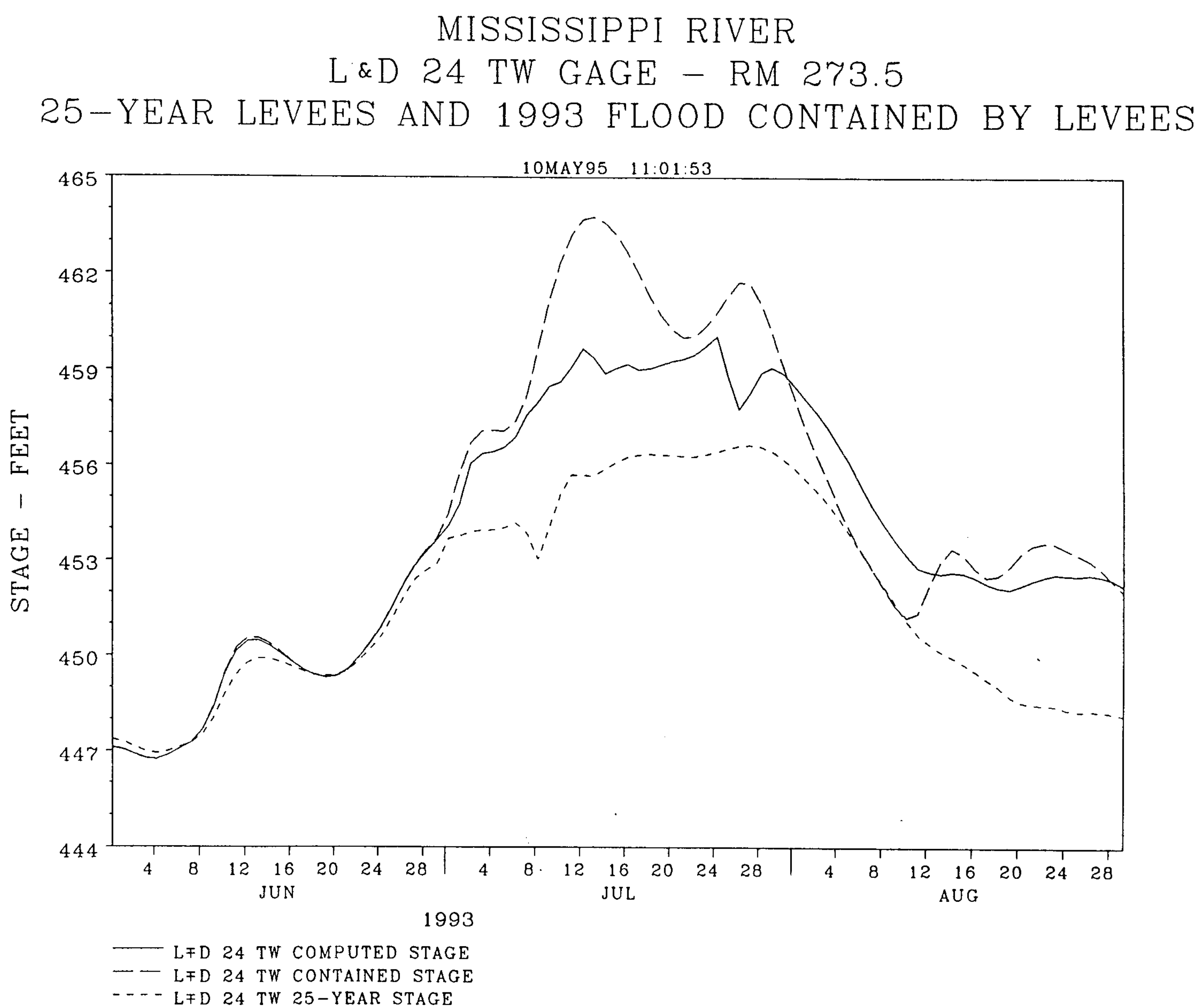

MISSISSIPPI RIVER
L&D 24 TW GAGE - RM 273.5
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES
10MAY95 11:01:53
STAGE - FEET
465
462
459
456
453
450
447
444
4 8 12 16 20 24 28 4 8 12 16 20 24 28 4 8 12 16 20 24 28
JUN
JUL
AUG
1993
L+D 24 TW COMPUTED STAGE
L+D 24 TW CONTAINED STAGE
L+D 24 TW 25-YEAR STAGE

PLATE SL-4.3

MISSISSIPPI RIVER
L &D 24 TW GAGE - RM 273.5
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS

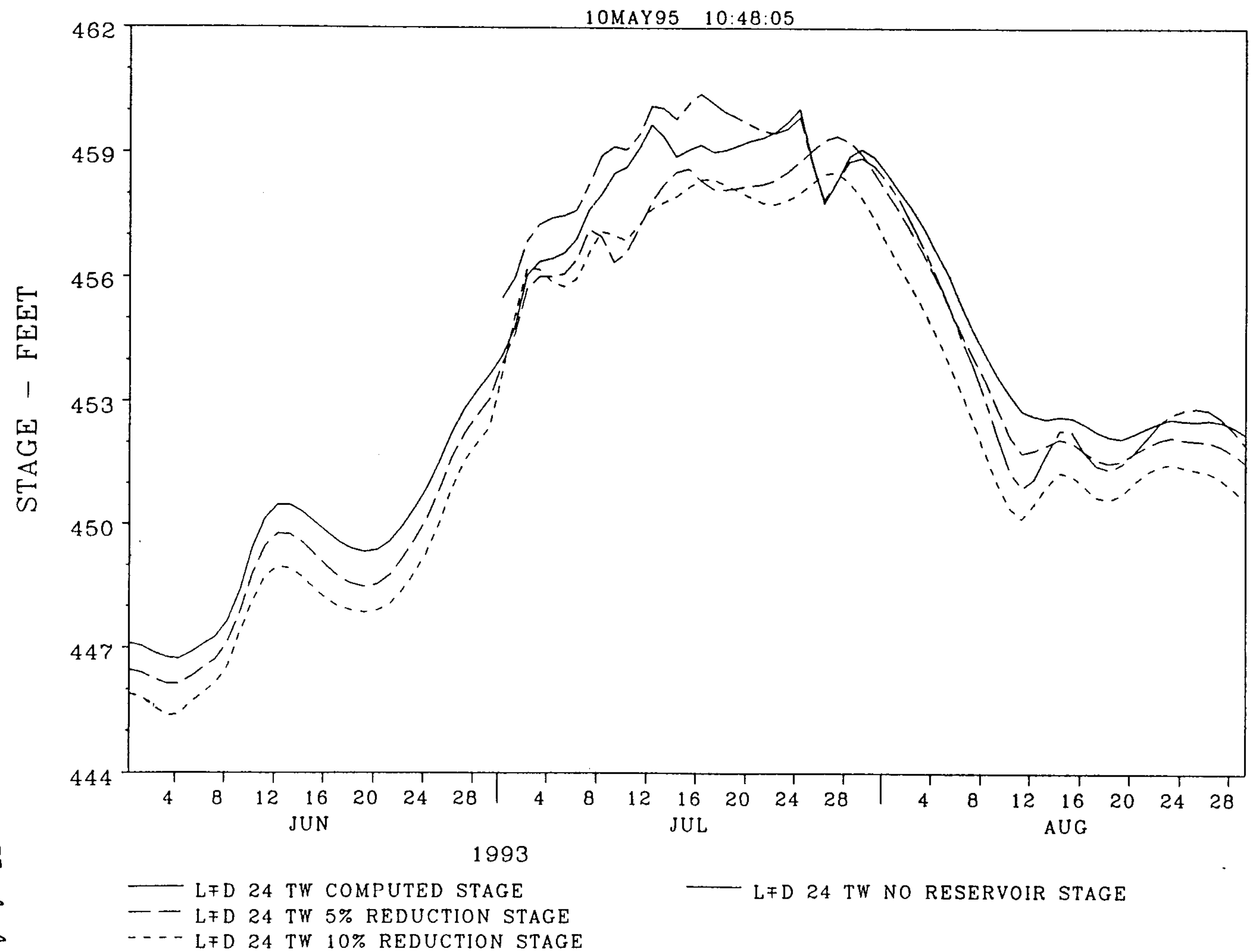


PLATE SL-4.4

MISSISSIPPI RIVER
L&D 24 TW GAGE - RM 273.5
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS

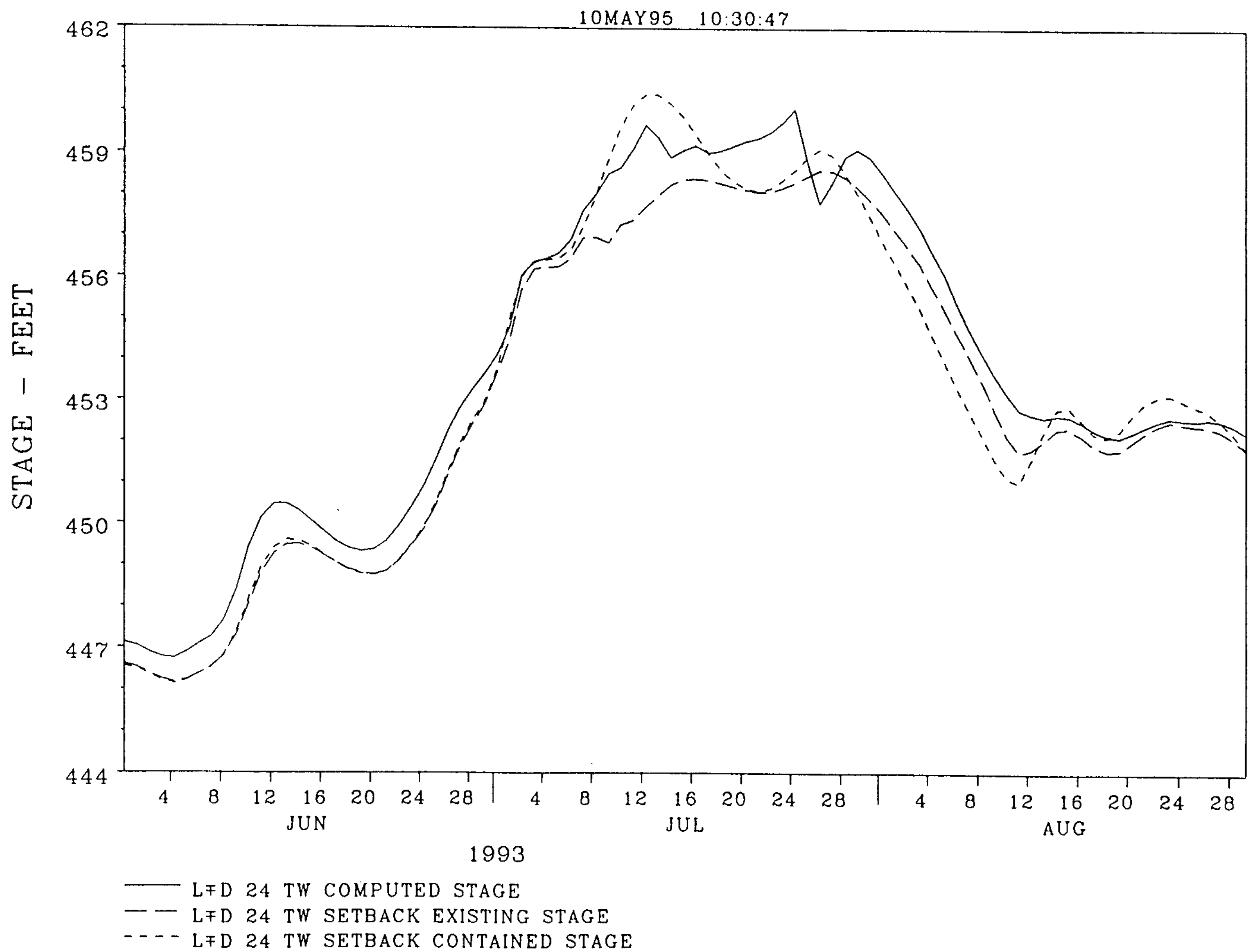


PLATE SL-4.5

MISSISSIPPI RIVER
L&D 25 TW GAGE - RM 241.2
COMPUTED VS OBSERVED STAGES -1993 FLOOD

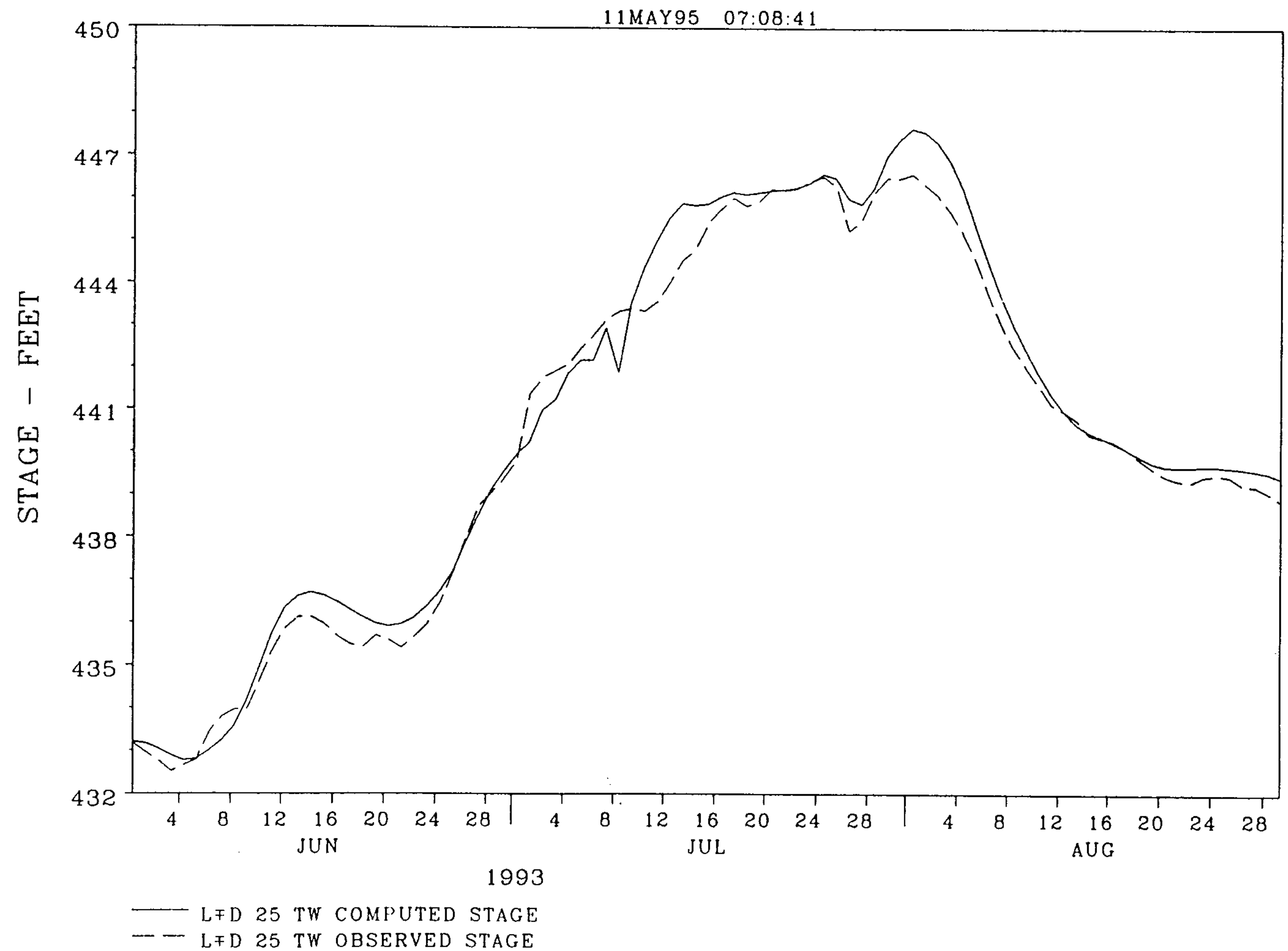


PLATE SL-5.1

MISSISSIPPI RIVER
L&D 25 TW GAGE - RM 241.2
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

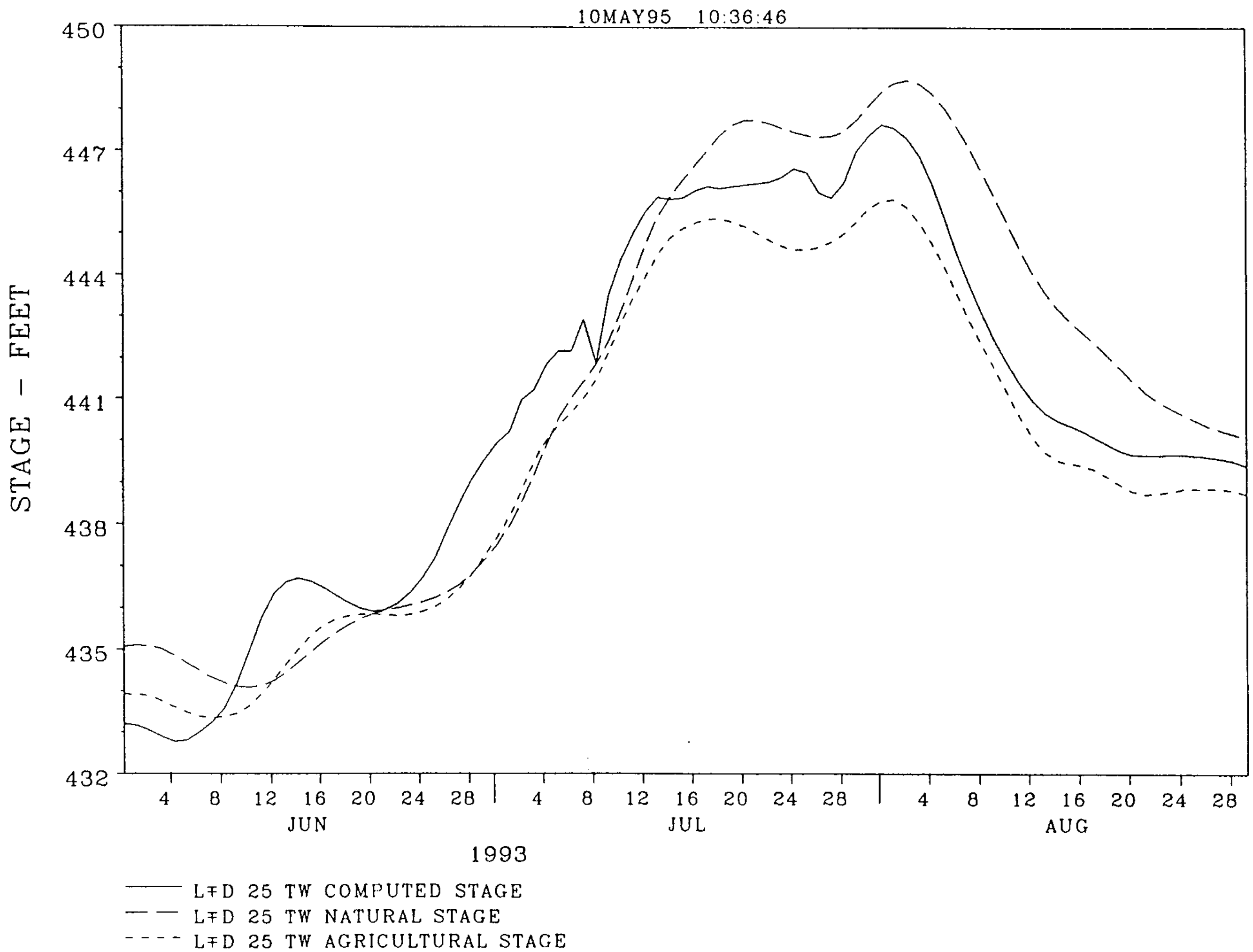


PLATE SL-5.2

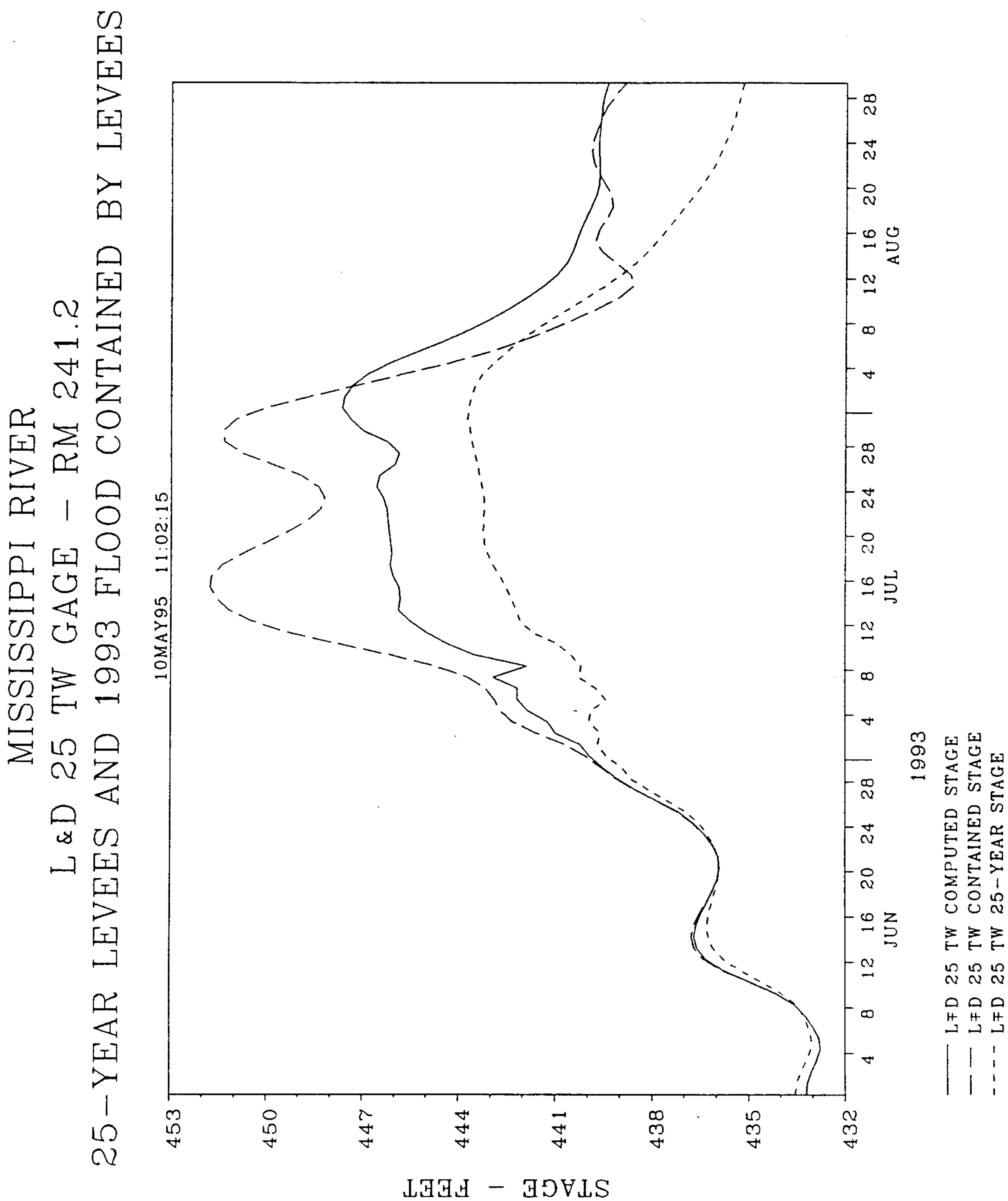

MISSISSIPPI RIVER
L &D 25 TW GAGE - RM 241.2
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES
10MAY95 11:02:15
STAGE - FEET
453
450
447
444
441
438
435
432
4 8 12 16 20 24 28
JUN
4 8 12 16 20 24 28
JUL
4 8 12 16 20 24 28
AUG
1993
L+D 25 TW COMPUTED STAGE
L+D 25 TW CONTAINED STAGE
L+D 25 TW 25-YEAR STAGE

PLATE SL-5.3

MISSISSIPPI RIVER
L &D 25 TW GAGE - RM 241.2
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS

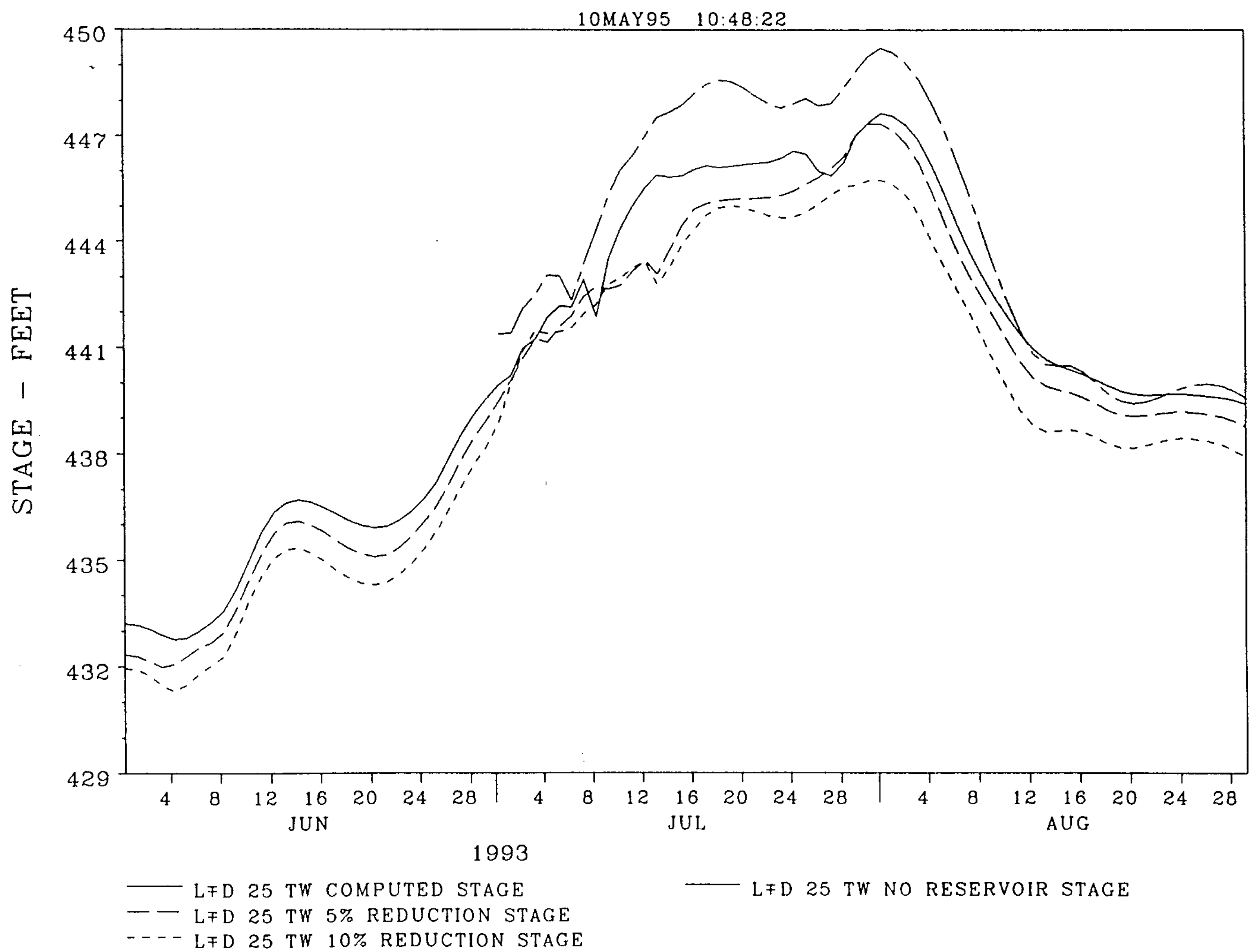
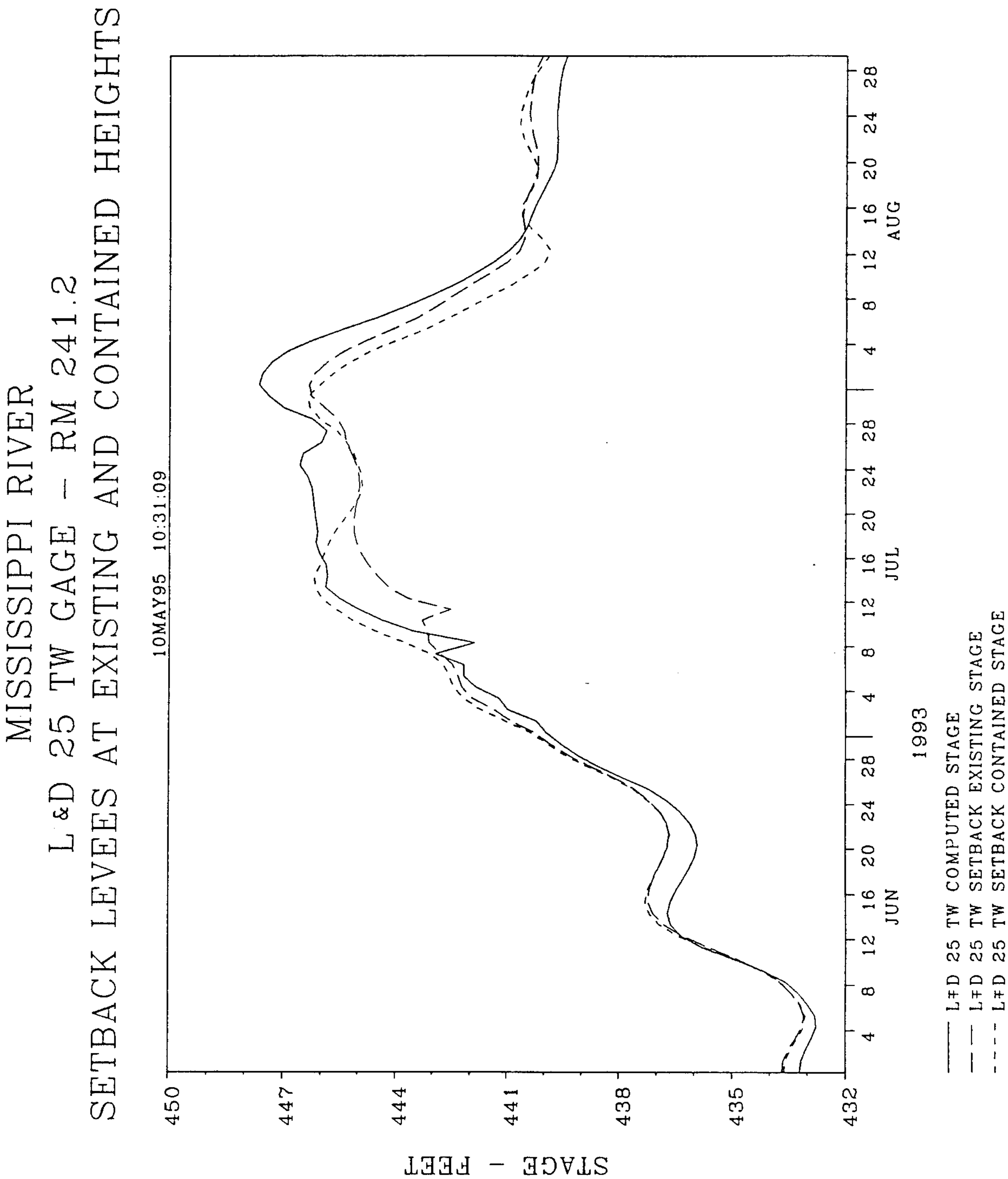


PLATE SL-5.4



MISSISSIPPI RIVER
L &D 25 TW GAGE - RM 241.2
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS
10MAY95 10:31:09
STAGE - FEET
450
447
444
441
438
435
432
4
8
12
16
20
24
28
4
8
12
16
20
24
28
4
8
12
16
20
24
28
JUN
JUL
AUG
1993
L+D 25 TW COMPUTED STAGE
L+D 25 TW SETBACK EXISTING STAGE
L+D 25 TW SETBACK CONTAINED STAGE

PLATE SL-5.5

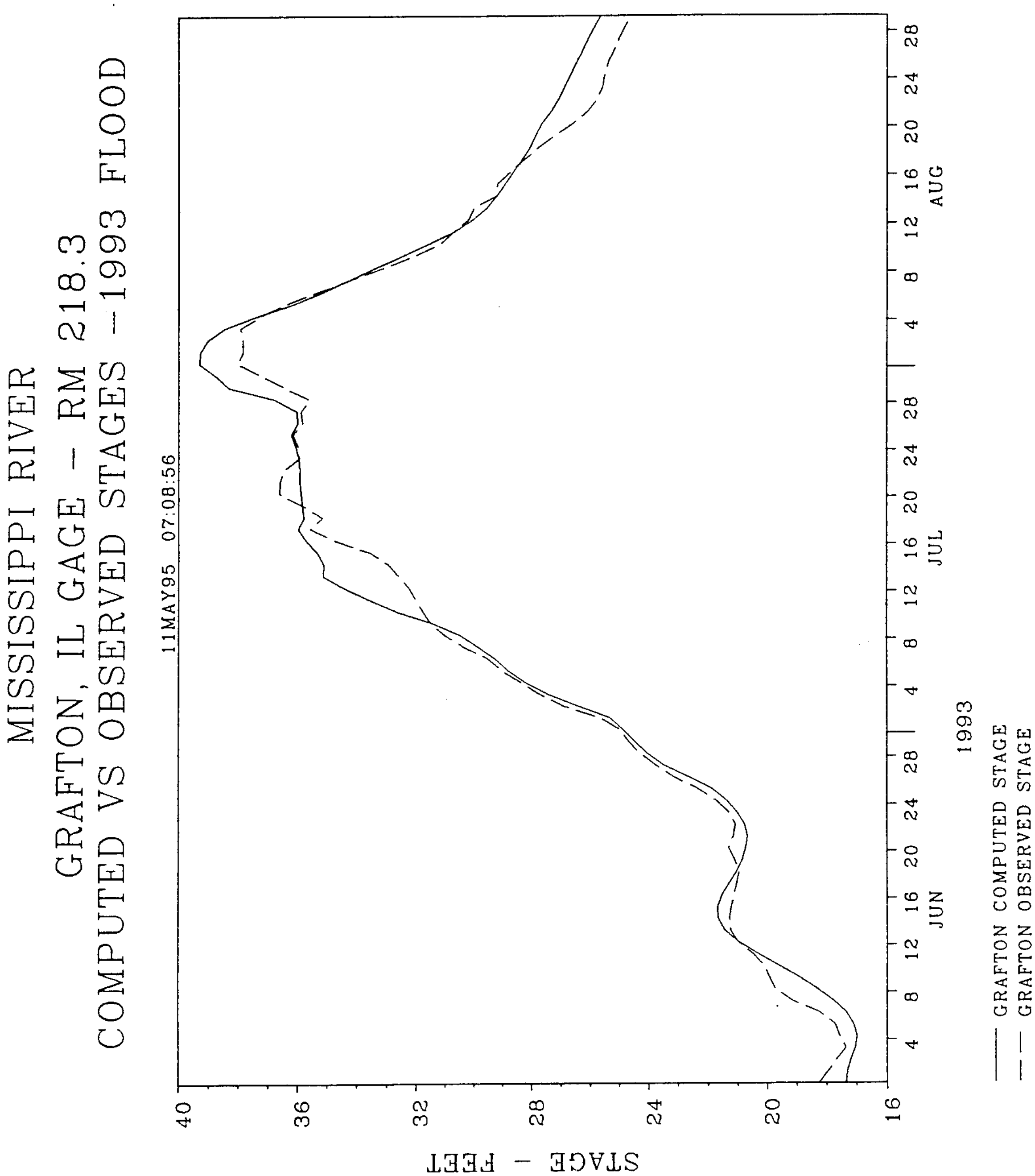

MISSISSIPPI RIVER
GRAFTON, IL GAGE - RM 218.3
COMPUTED VS OBSERVED STAGES -1993 FLOOD
11MAY95 07:08:56
STAGE - FEET
40
36
32
28
24
20
16
4 8 12 16 20 24 28
JUN
4 8 12 16 20 24 28
JUL
4 8 12 16 20 24 28
AUG
1993
GRAFTON COMPUTED STAGE
GRAFTON OBSERVED STAGE

PLATE SL-6.1

MISSISSIPPI RIVER
GRAFTON, IL GAGE – RM 218.3
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

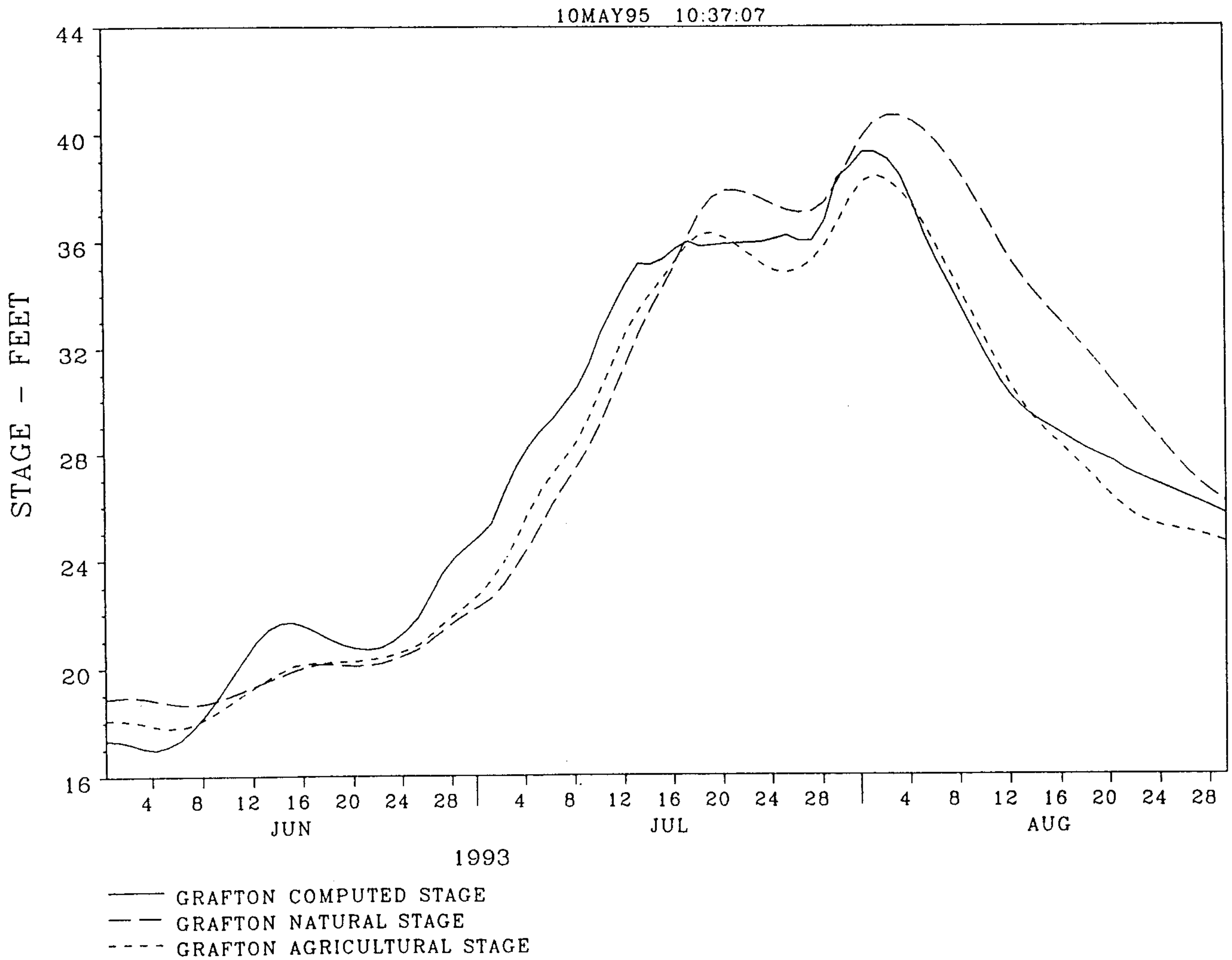


PLATE SL-6.2

MISSISSIPPI RIVER
GRAFTON, IL GAGE – RM 218.3
25–YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES

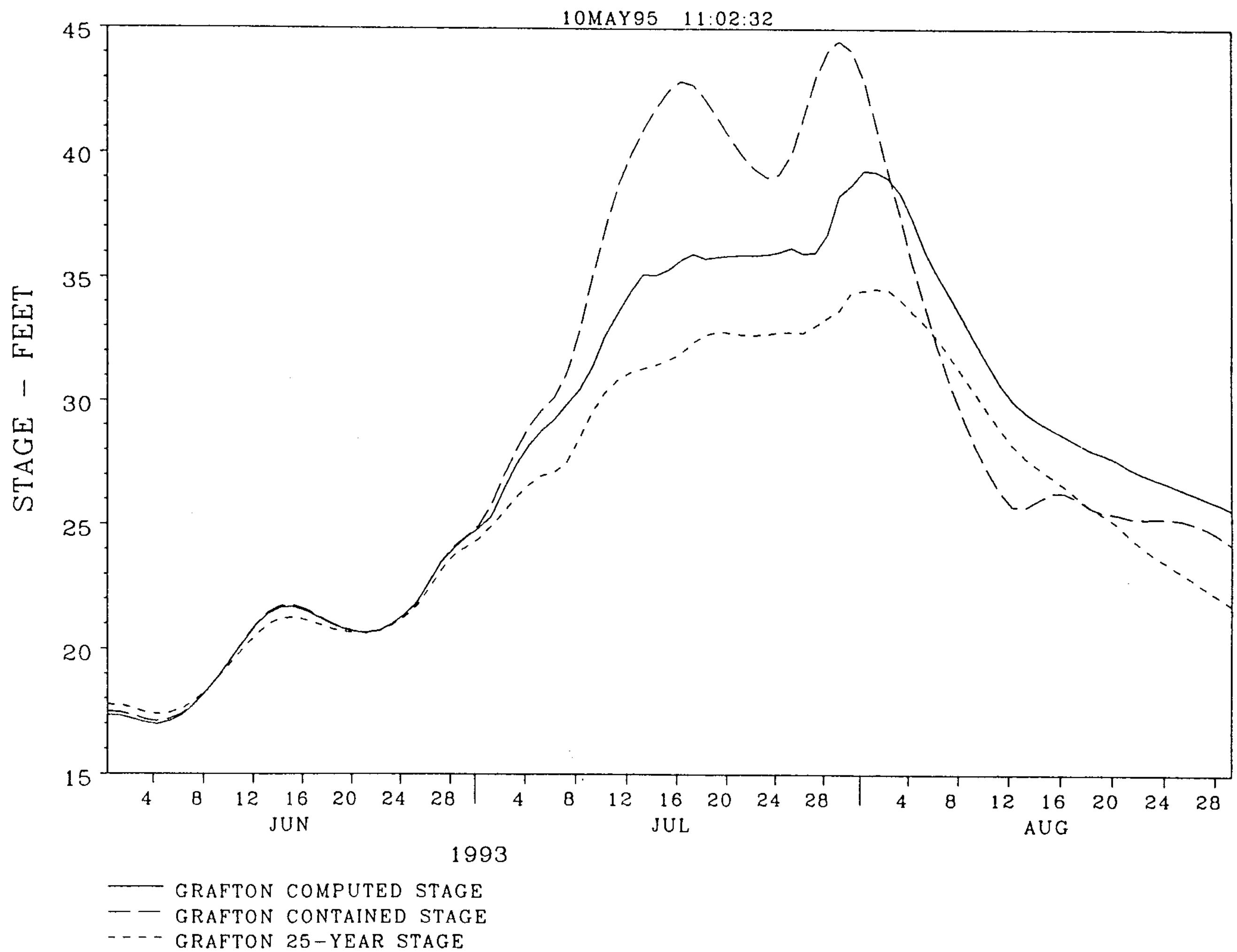


PLATE SL-6.3

MISSISSIPPI RIVER
GRAFTON, IL GAGE – RM 218.3
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS

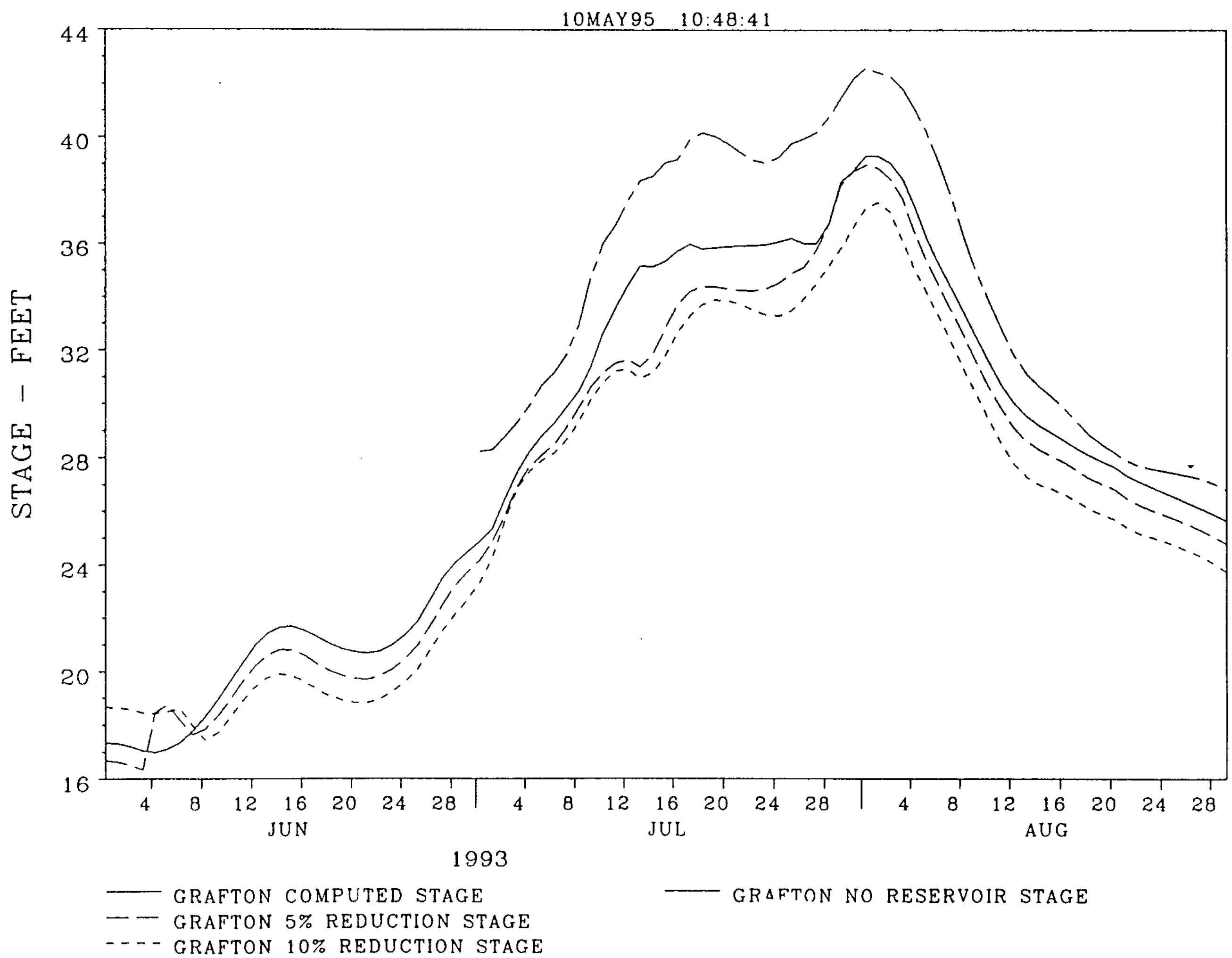
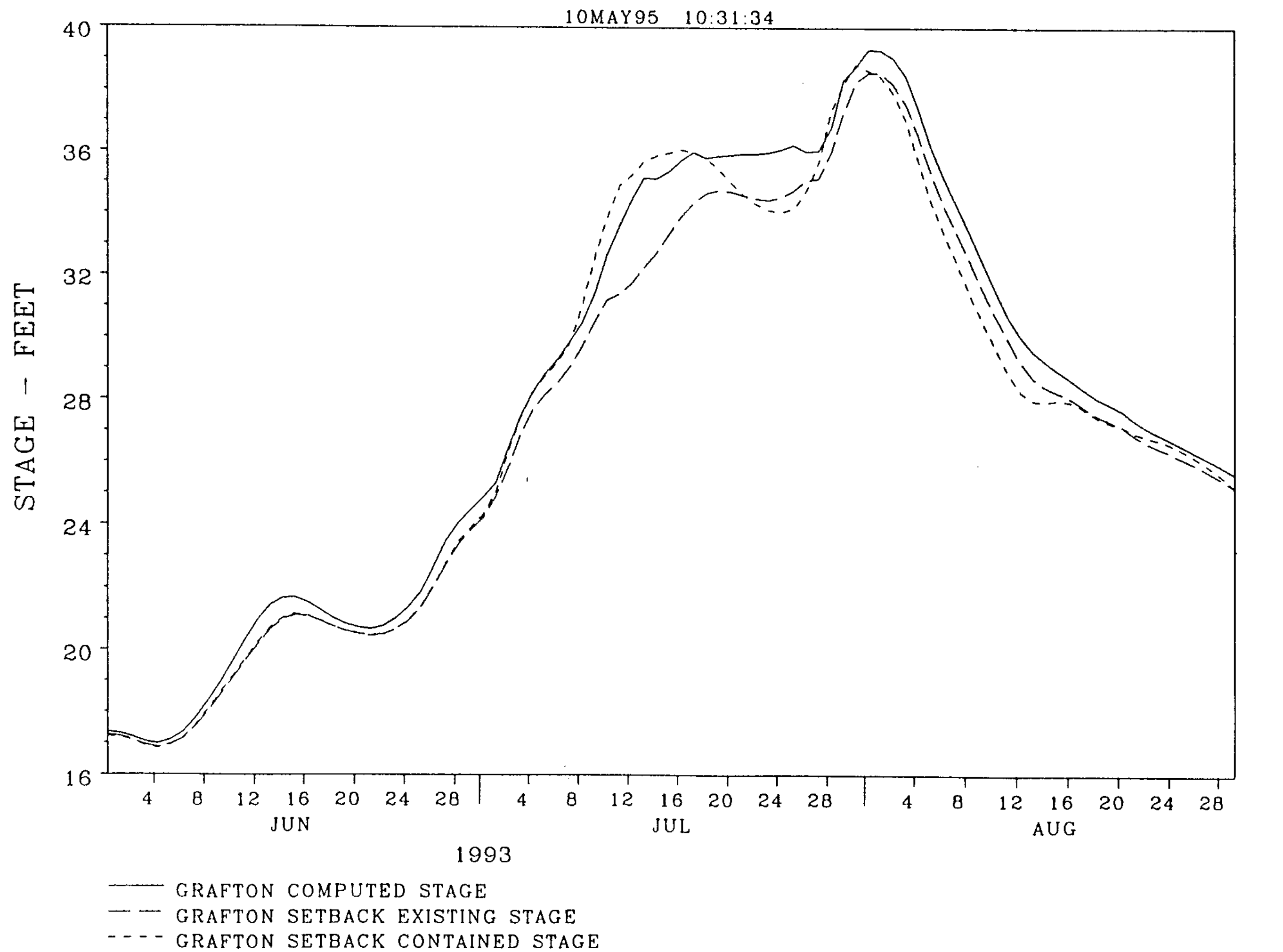
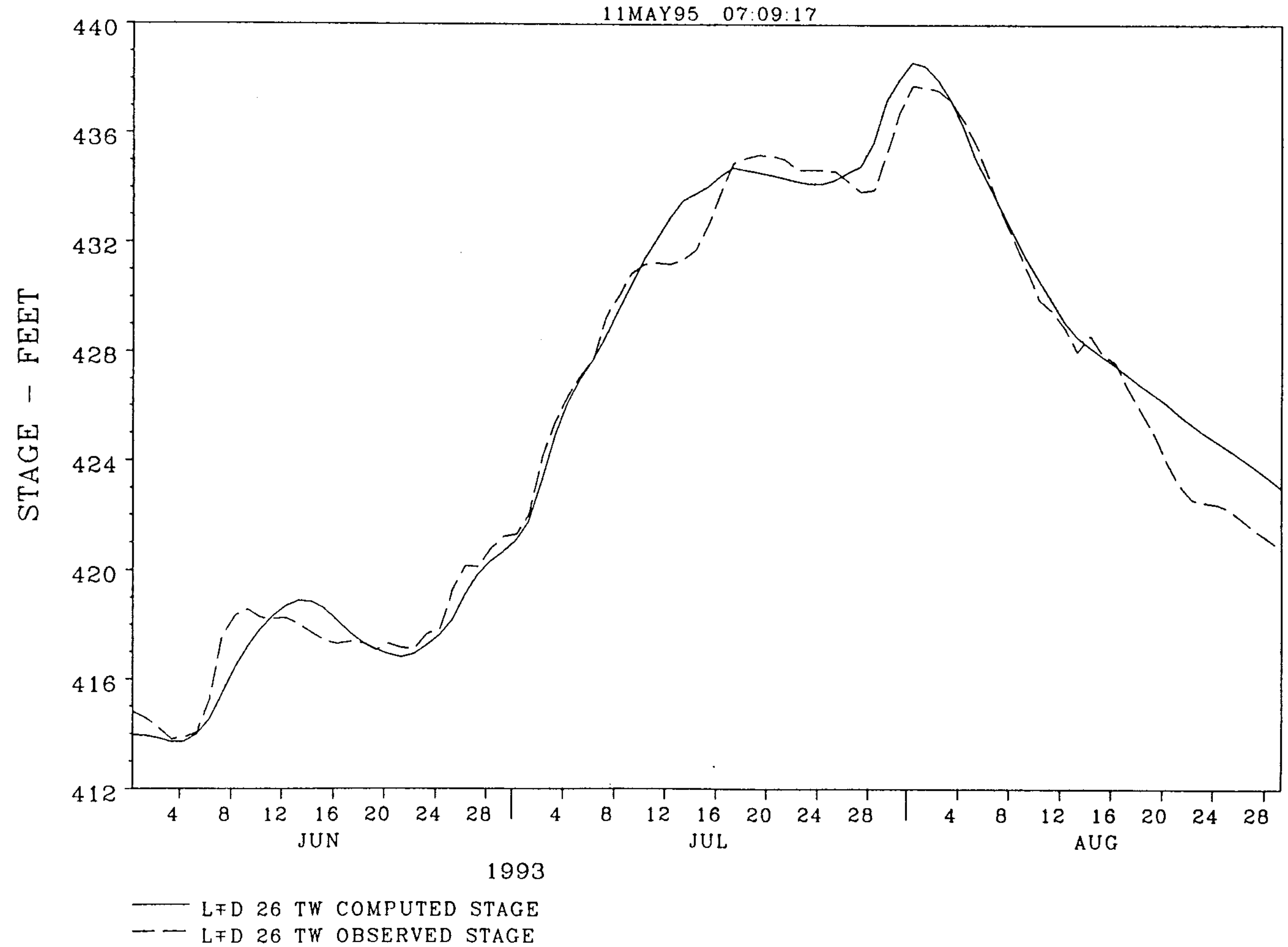


PLATE SL-6.4

MISSISSIPPI RIVER
GRAFTON, IL GAGE – RM 218.3
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS



MISSISSIPPI RIVER
L &D 26 TW GAGE - RM 199.9
COMPUTED VS OBSERVED STAGES -1993 FLOOD



MISSISSIPPI RIVER
L &D 26 TW GAGE - RM 199.9
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

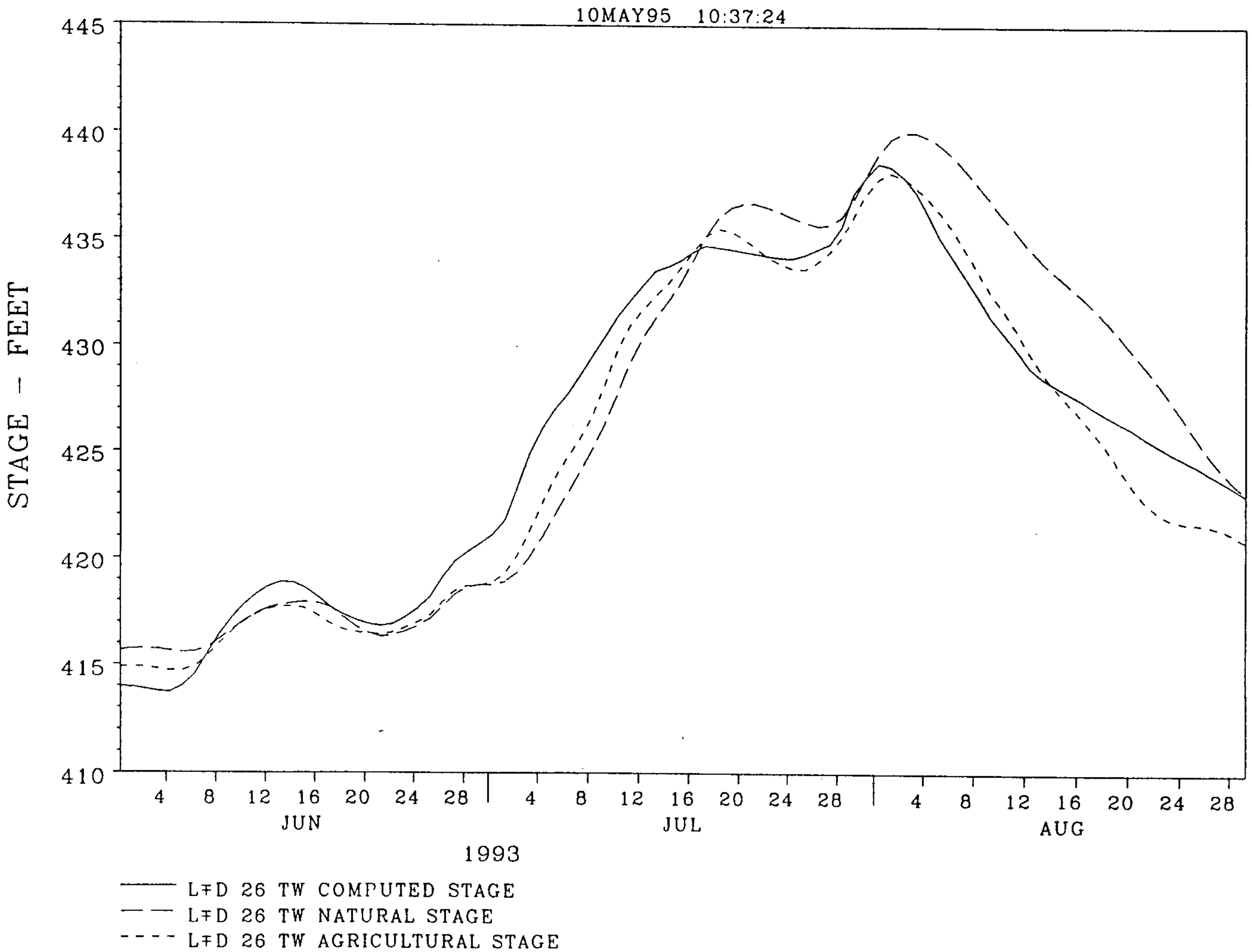


PLATE SL-7.2

MISSISSIPPI RIVER
L&D 26 TW GAGE - RM 199.9
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES

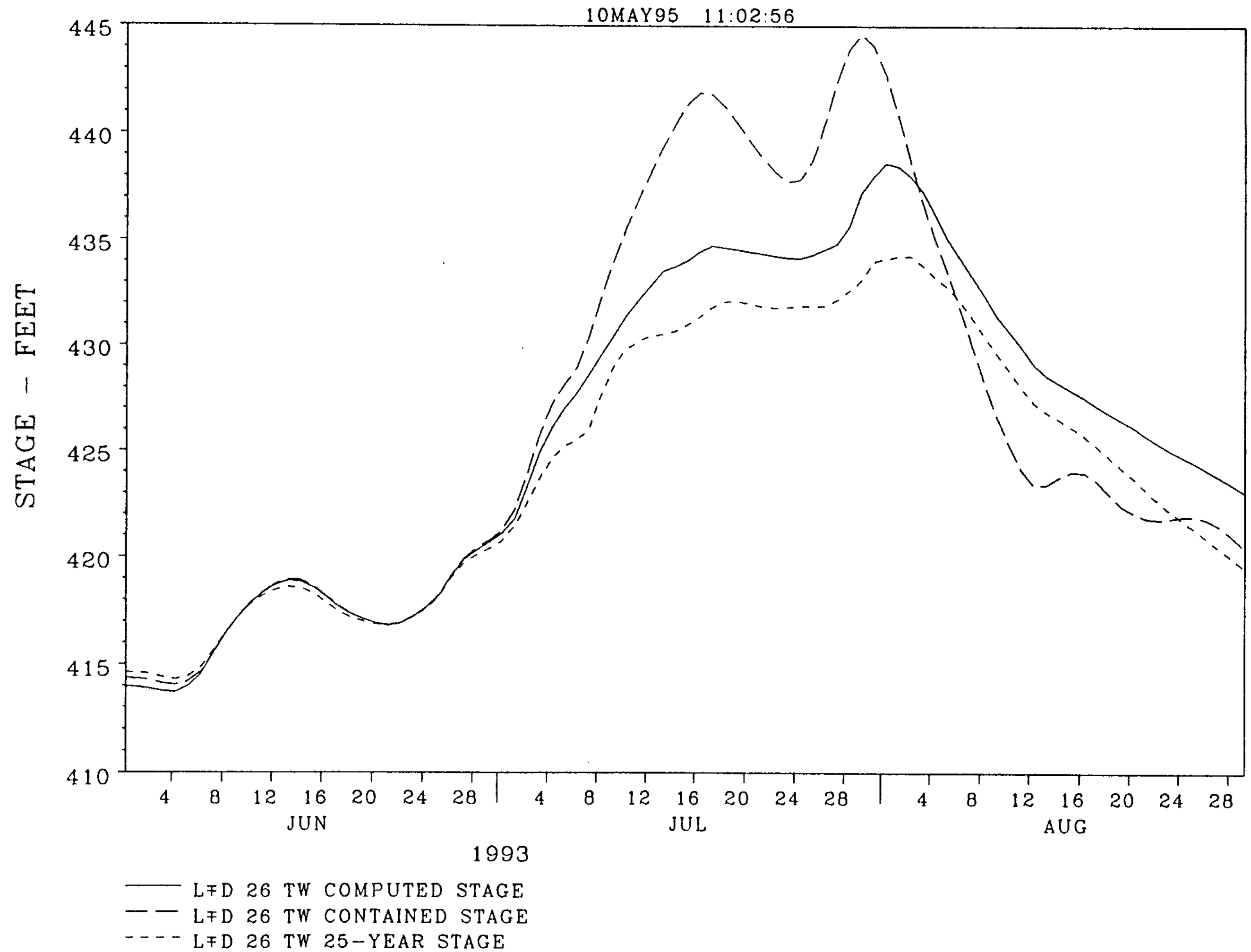


PLATE SL-7.3

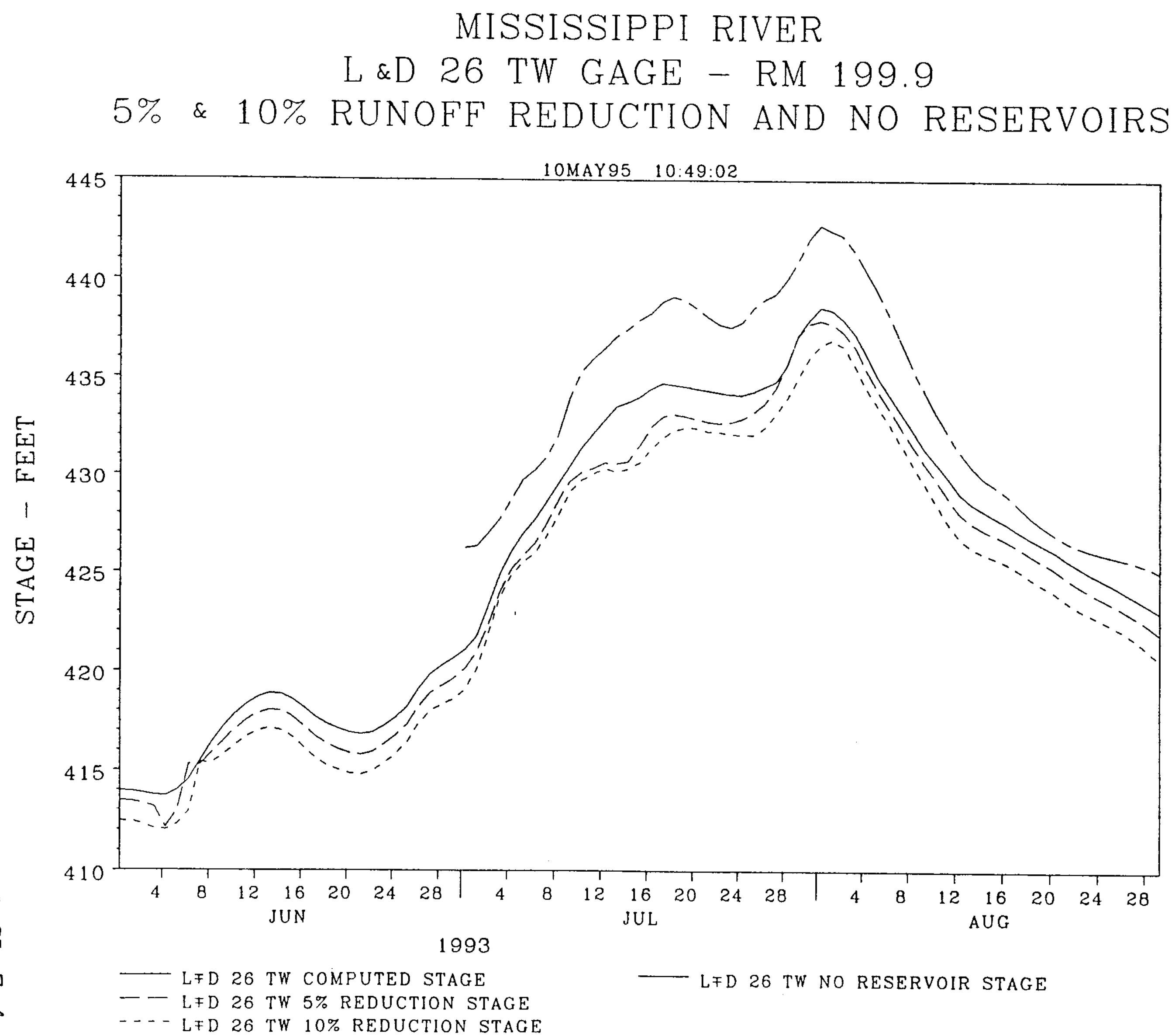
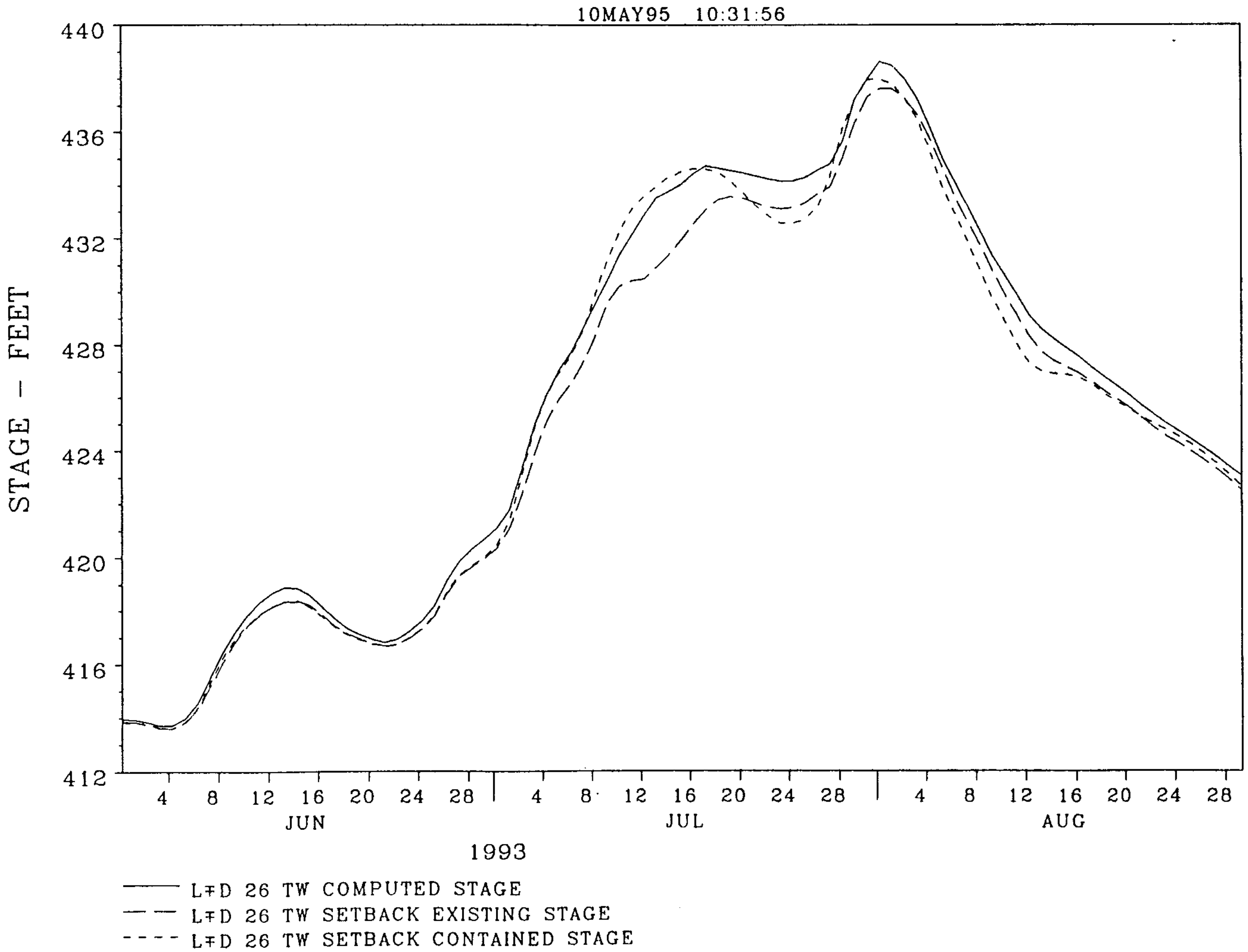

MISSISSIPPI RIVER
L &D 26 TW GAGE - RM 199.9
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS
10MAY95 10:49:02
STAGE - FEET
445
440
435
430
425
420
415
410
4 8 12 16 20 24 28
JUN
4 8 12 16 20 24 28
JUL
4 8 12 16 20 24 28
AUG
1993
L+D 26 TW COMPUTED STAGE
L+D 26 TW 5% REDUCTION STAGE
L+D 26 TW 10% REDUCTION STAGE
L+D 26 TW NO RESERVOIR STAGE

PLATE SL-7.4

MISSISSIPPI RIVER
L&D 26 TW GAGE – RM 199.9
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS



MISSISSIPPI RIVER
ST. LOUIS, MO GAGE – RM 179.6
COMPUTED VS OBSERVED STAGES –1993 FLOOD

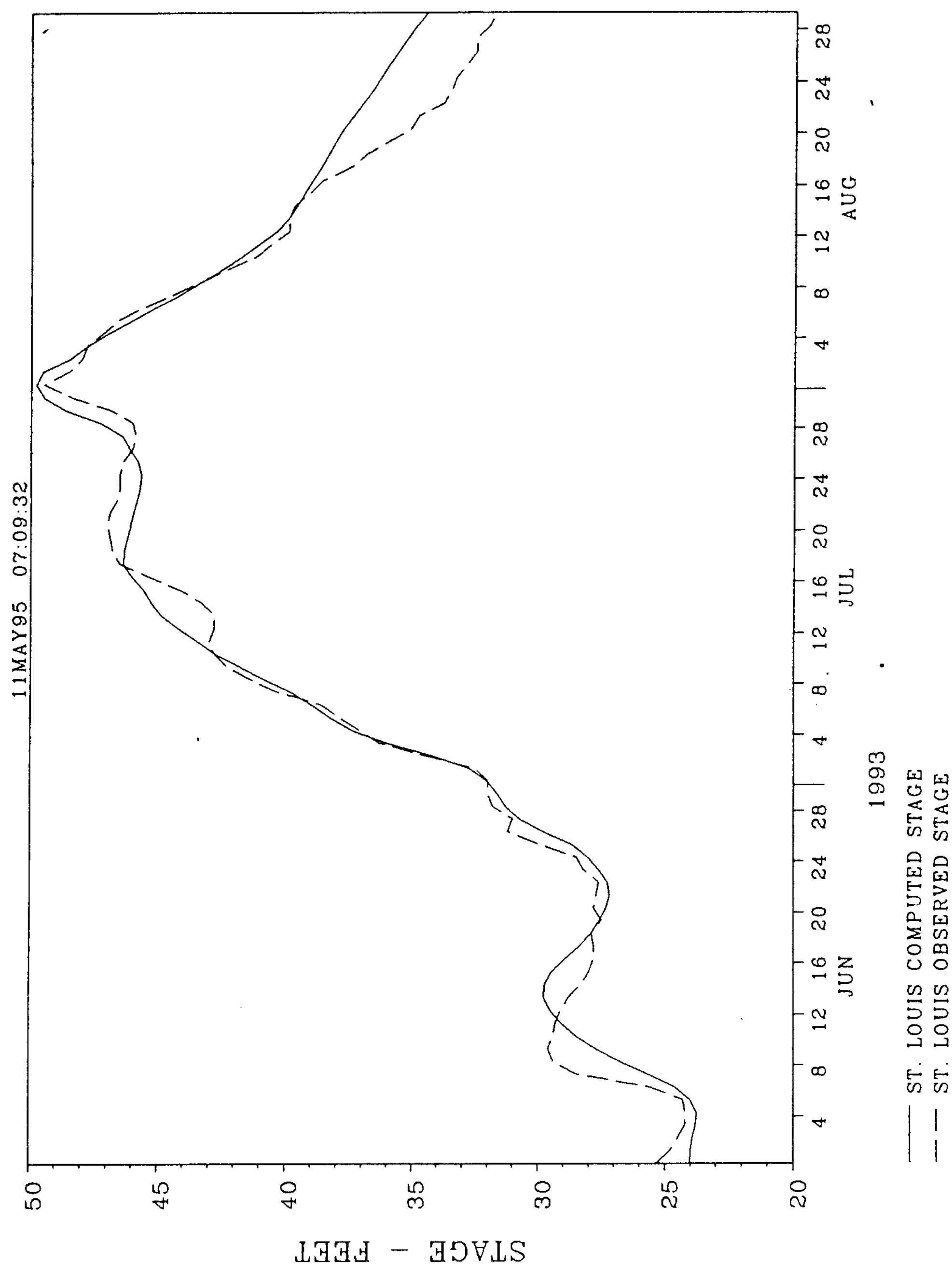


PLATE SL-8.1

MISSISSIPPI RIVER
ST. LOUIS, MO GAGE – RM 179.6
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

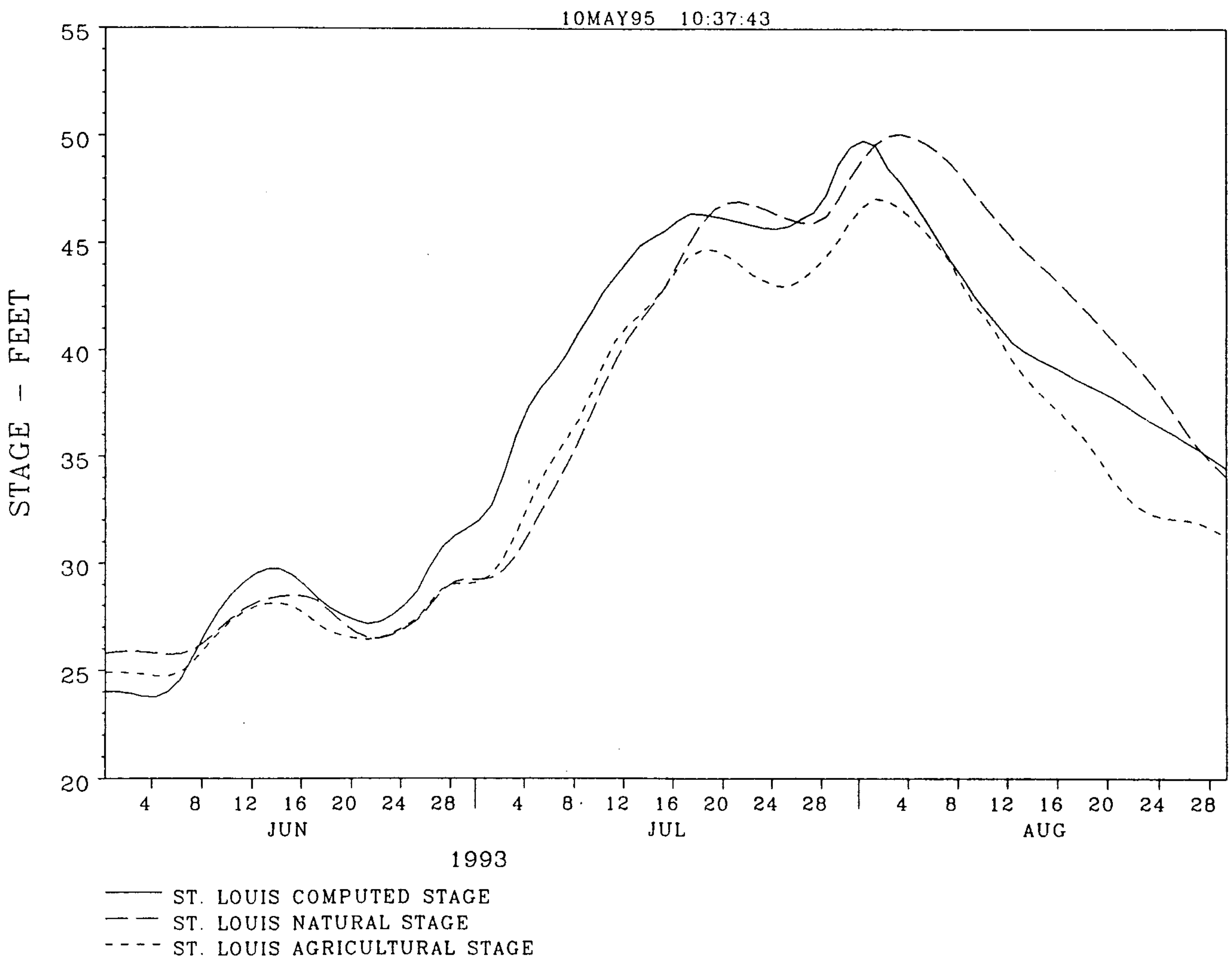
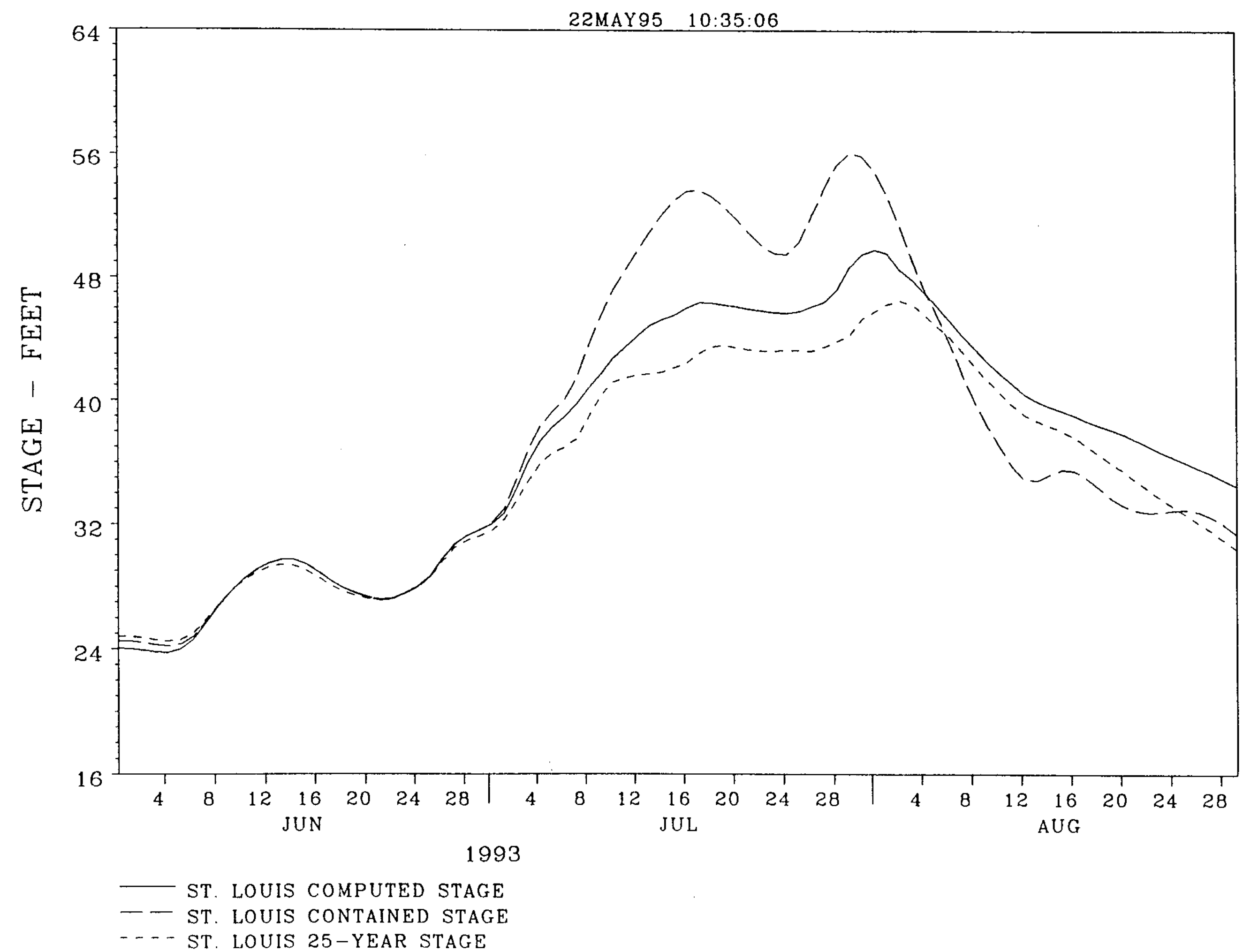
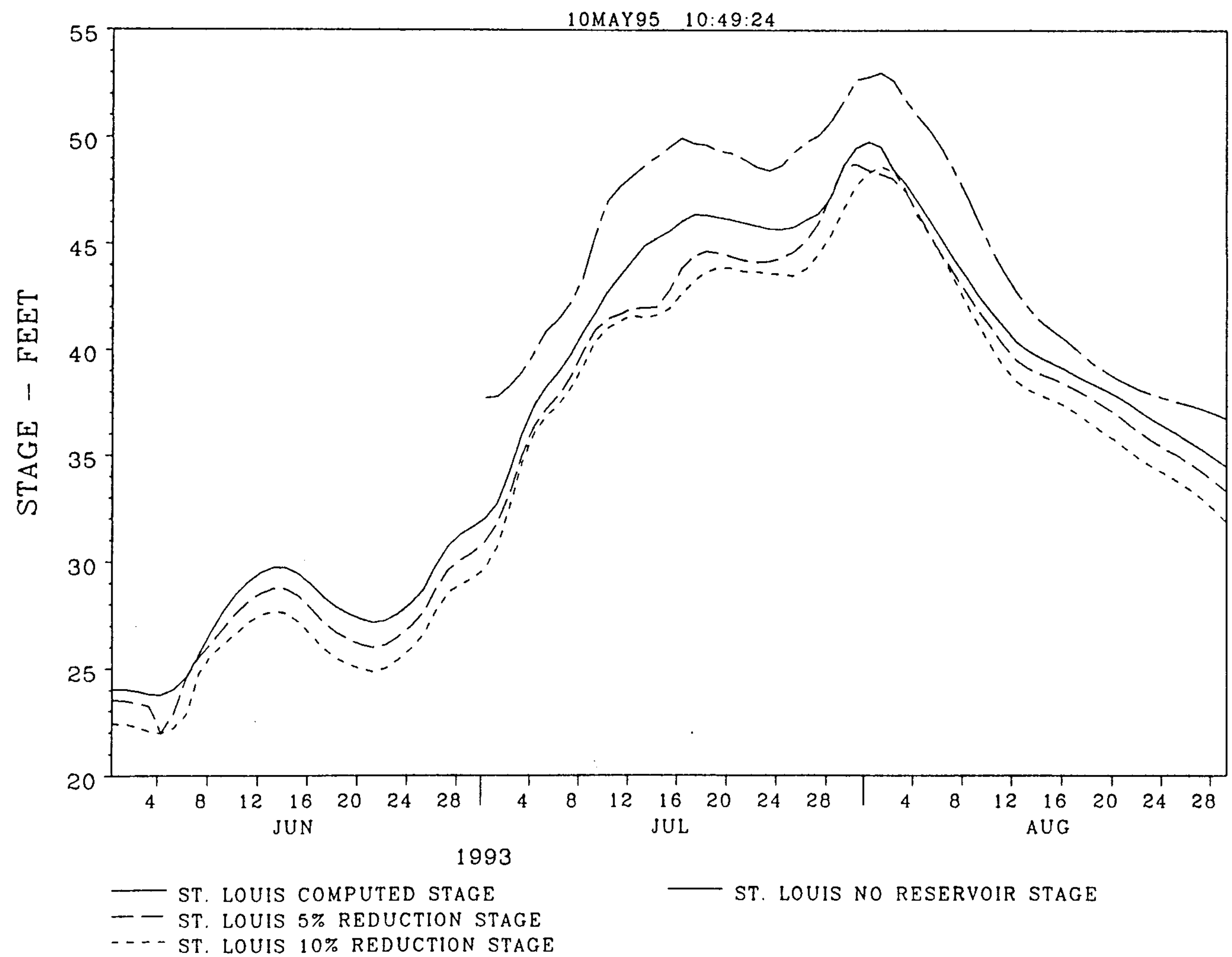


PLATE SL-8.2

MISSISSIPPI RIVER
ST. LOUIS, MO GAGE – RM 179.6
25–YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



MISSISSIPPI RIVER
ST. LOUIS, MO GAGE - RM 179.6
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS



MISSISSIPPI RIVER
ST. LOUIS, MO GAGE – RM 179.6
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS

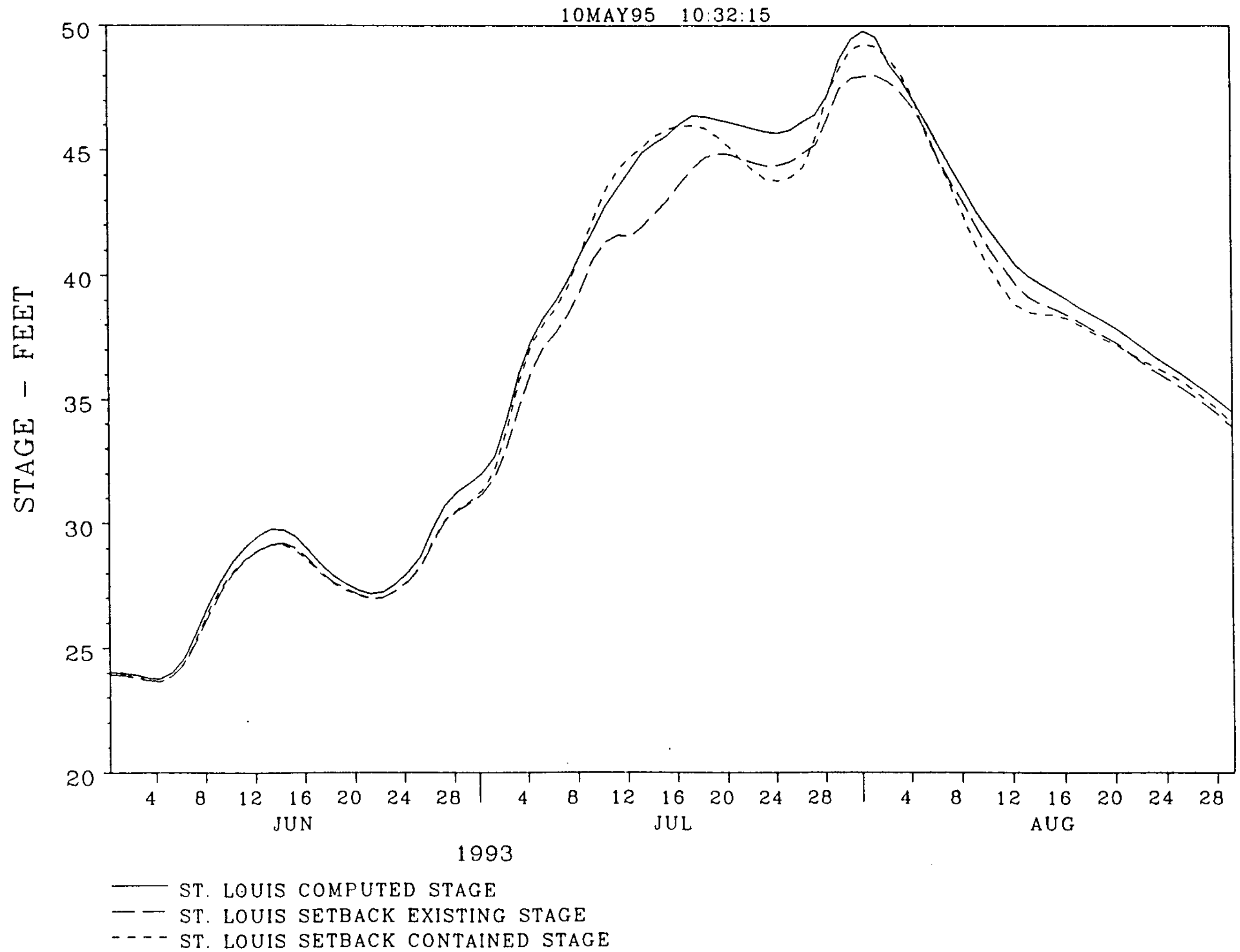


PLATE SL-8.5

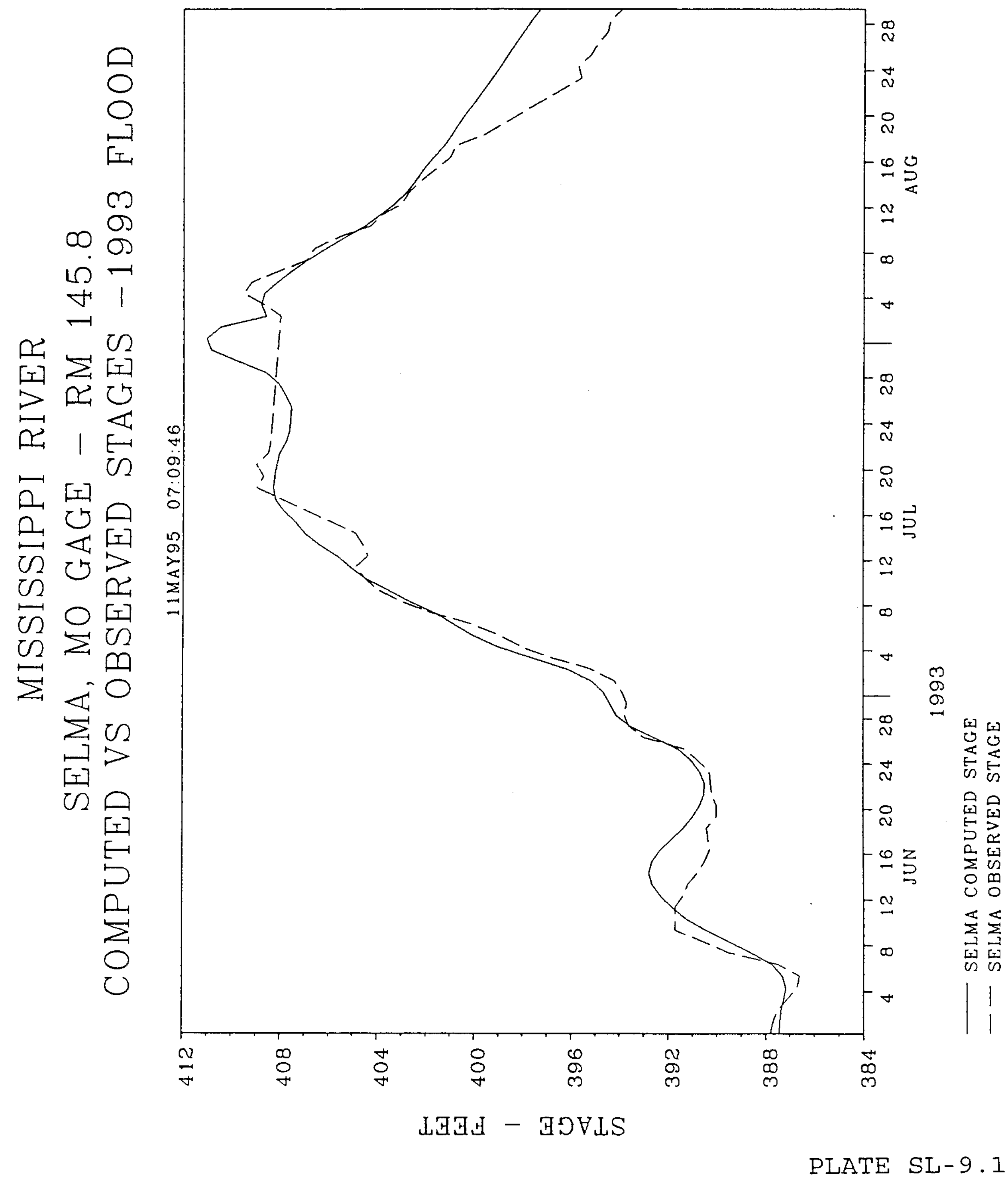

MISSISSIPPI RIVER
SELMA, MO GAGE – RM 145.8
COMPUTED VS OBSERVED STAGES –1993 FLOOD
11MAY95 07:09:46
STAGE – FEET
412
408
404
400
396
392
388
384
4
8
12
16
20
24
28
JUN
JUL
AUG
1993
SELMA COMPUTED STAGE
SELMA OBSERVED STAGE

PLATE SL-9.1

MISSISSIPPI RIVER
SELMA, MO GAGE – RM 145.8
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

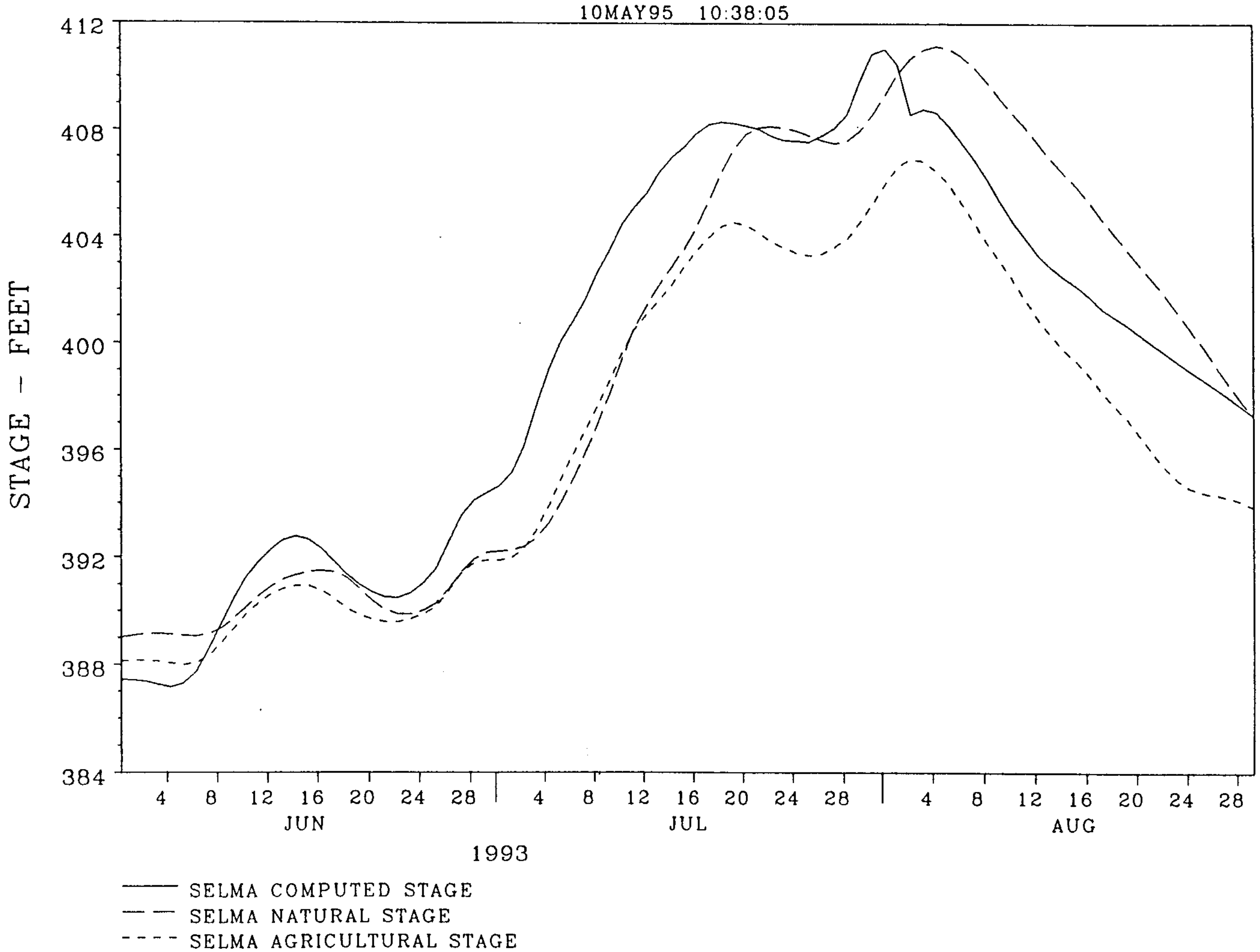


PLATE SL-9.2

MISSISSIPPI RIVER
SELMA, MO GAGE - RM 145.8
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES

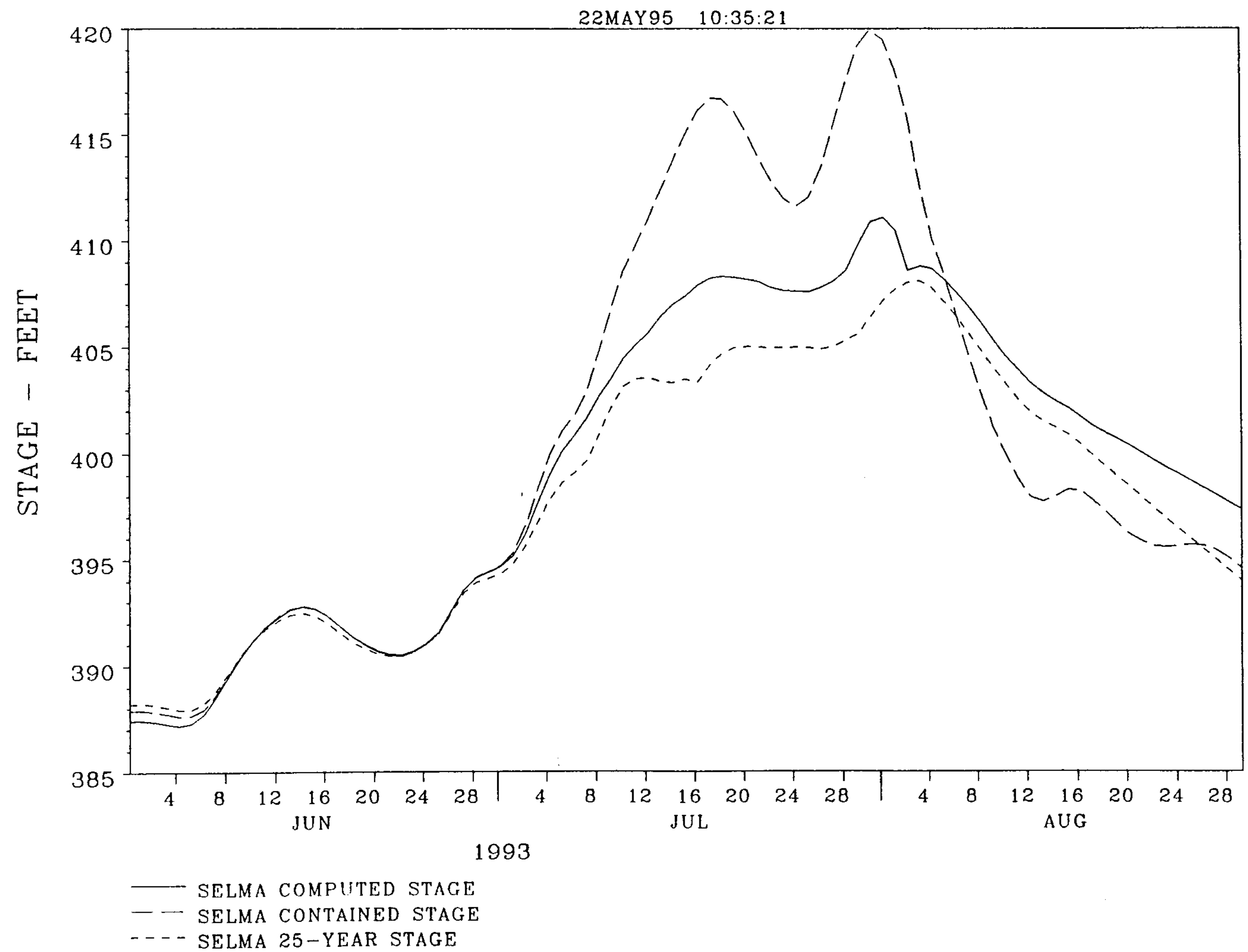


PLATE SL-9.3

MISSISSIPPI RIVER
SELMA, MO GAGE – RM 145.8
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS

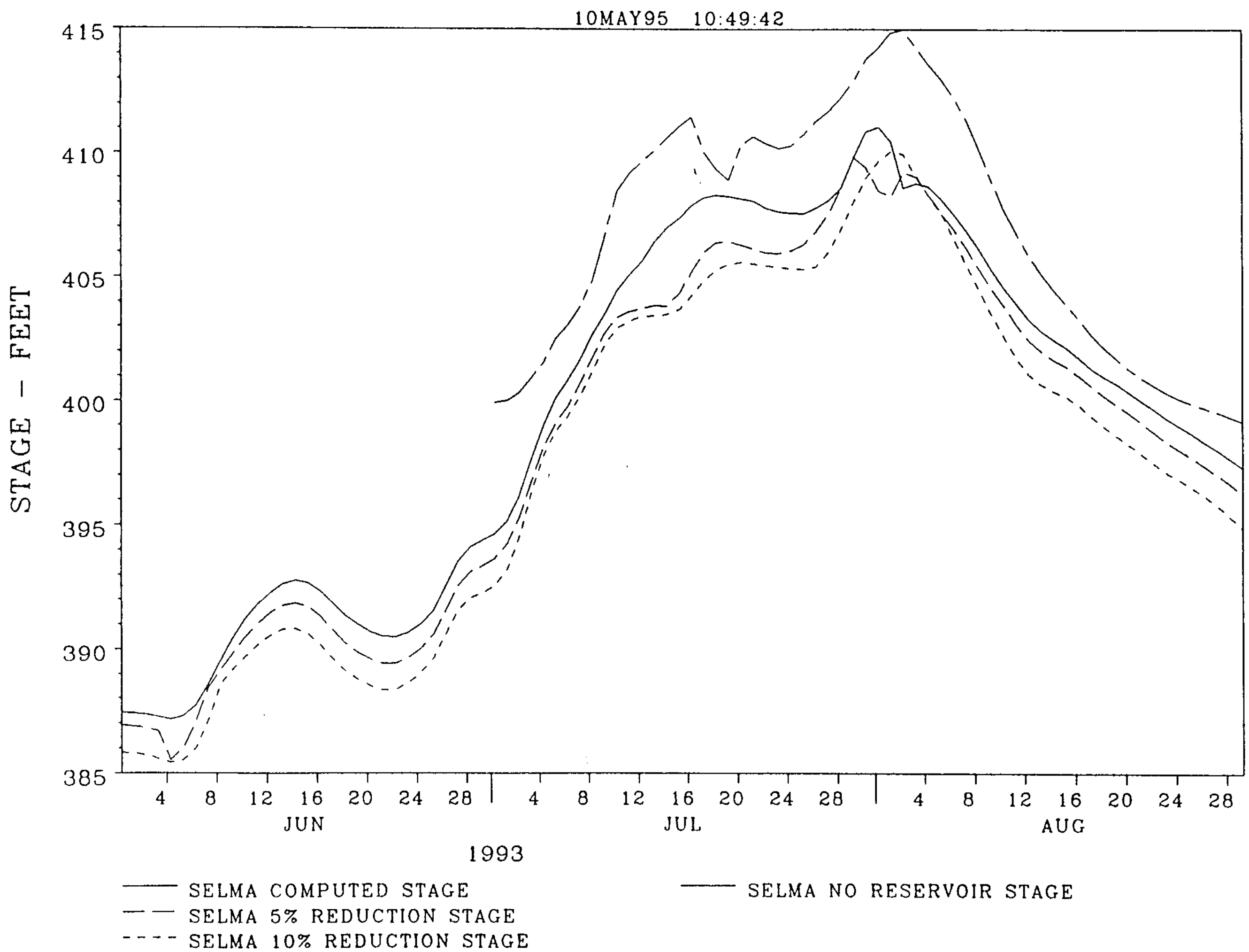


PLATE SL-9.4

MISSISSIPPI RIVER
SELMA, MO GAGE - RM 145.8
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS

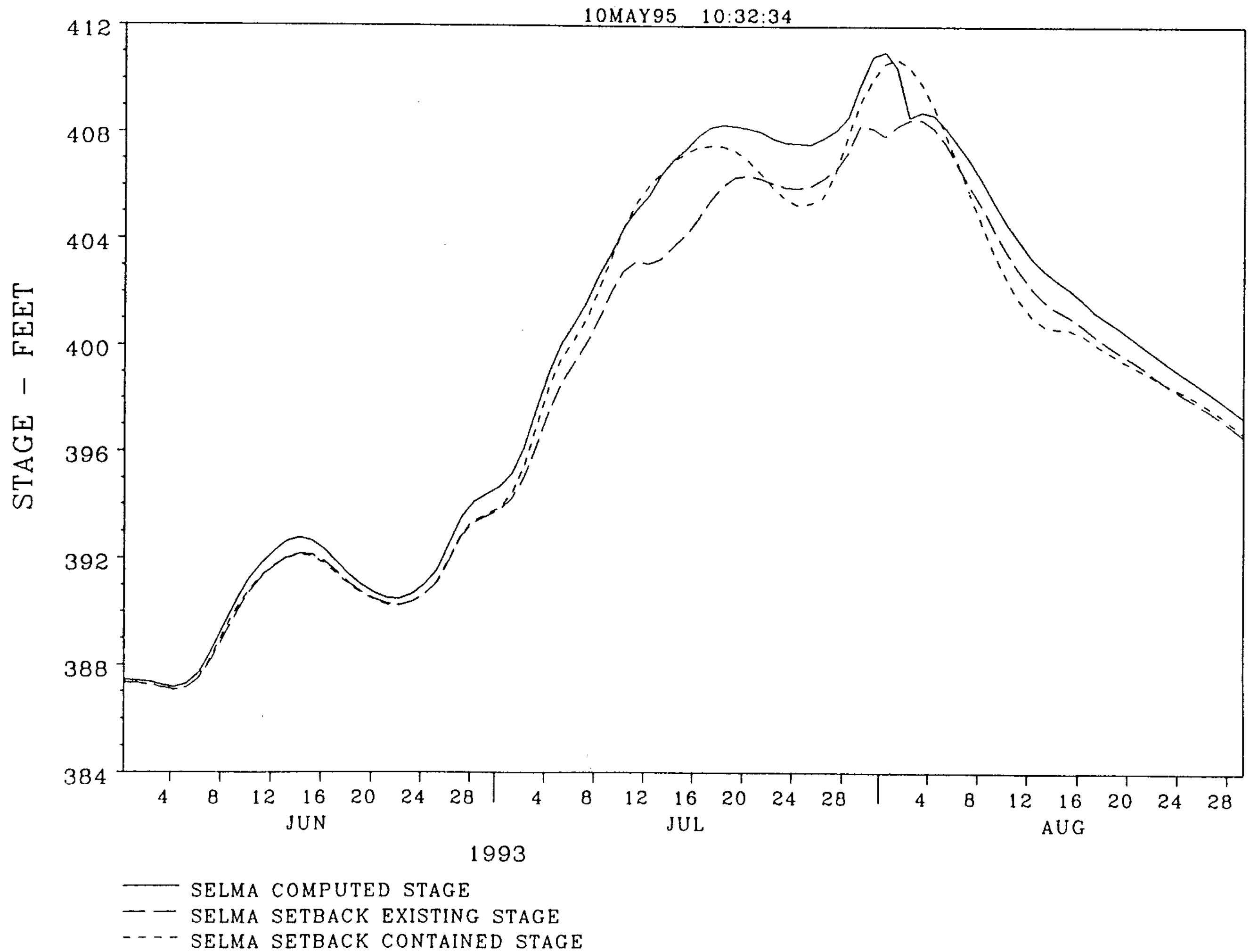


PLATE SL-9.5

MISSISSIPPI RIVER
CHESTER, IL GAGE – RM 109.9
COMPUTED VS OBSERVED STAGES –1993 FLOOD

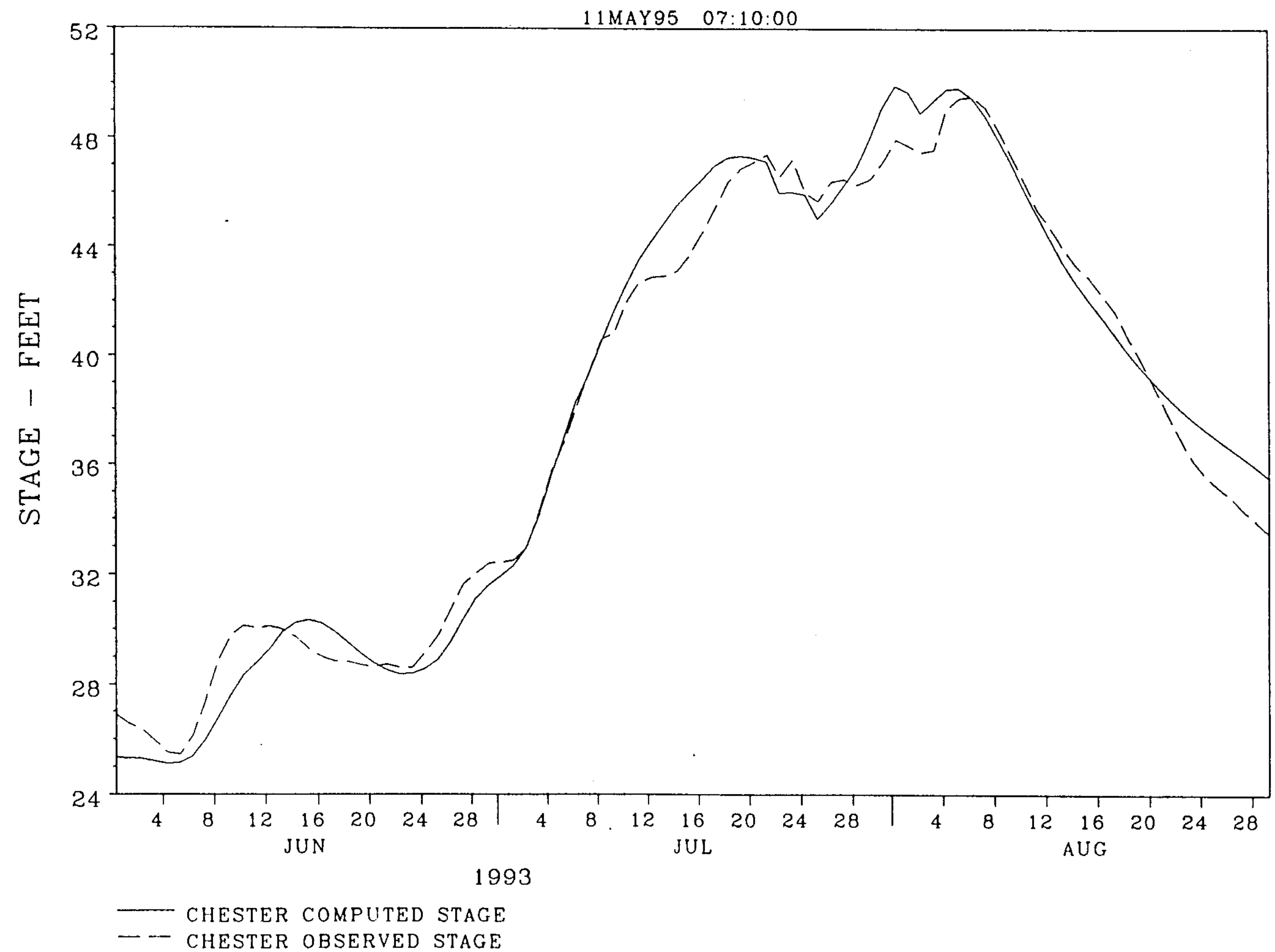


PLATE SL-10.1

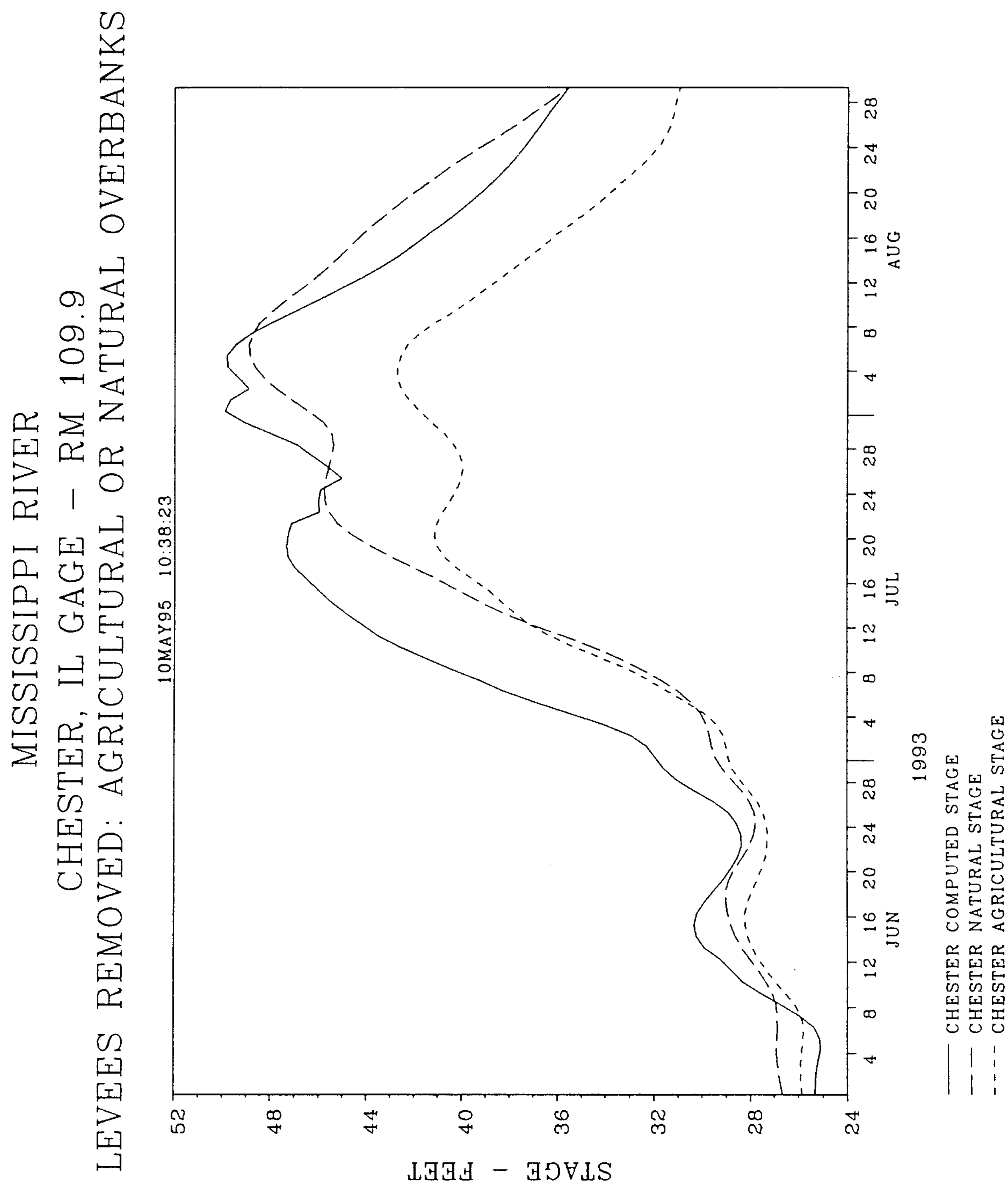

MISSISSIPPI RIVER
CHESTER, IL GAGE - RM 109.9
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS
10MAY95 10:38:23
STAGE - FEET
52
48
44
40
36
32
28
24
4 8 12 16 20 24 28 4 8 12 16 20 24 28 4 8 12 16 20 24 28
JUN
JUL
AUG
1993
CHESTER COMPUTED STAGE
CHESTER NATURAL STAGE
CHESTER AGRICULTURAL STAGE

PLATE SL-10.2

MISSISSIPPI RIVER
CHESTER, IL GAGE – RM 109.9
25–YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES

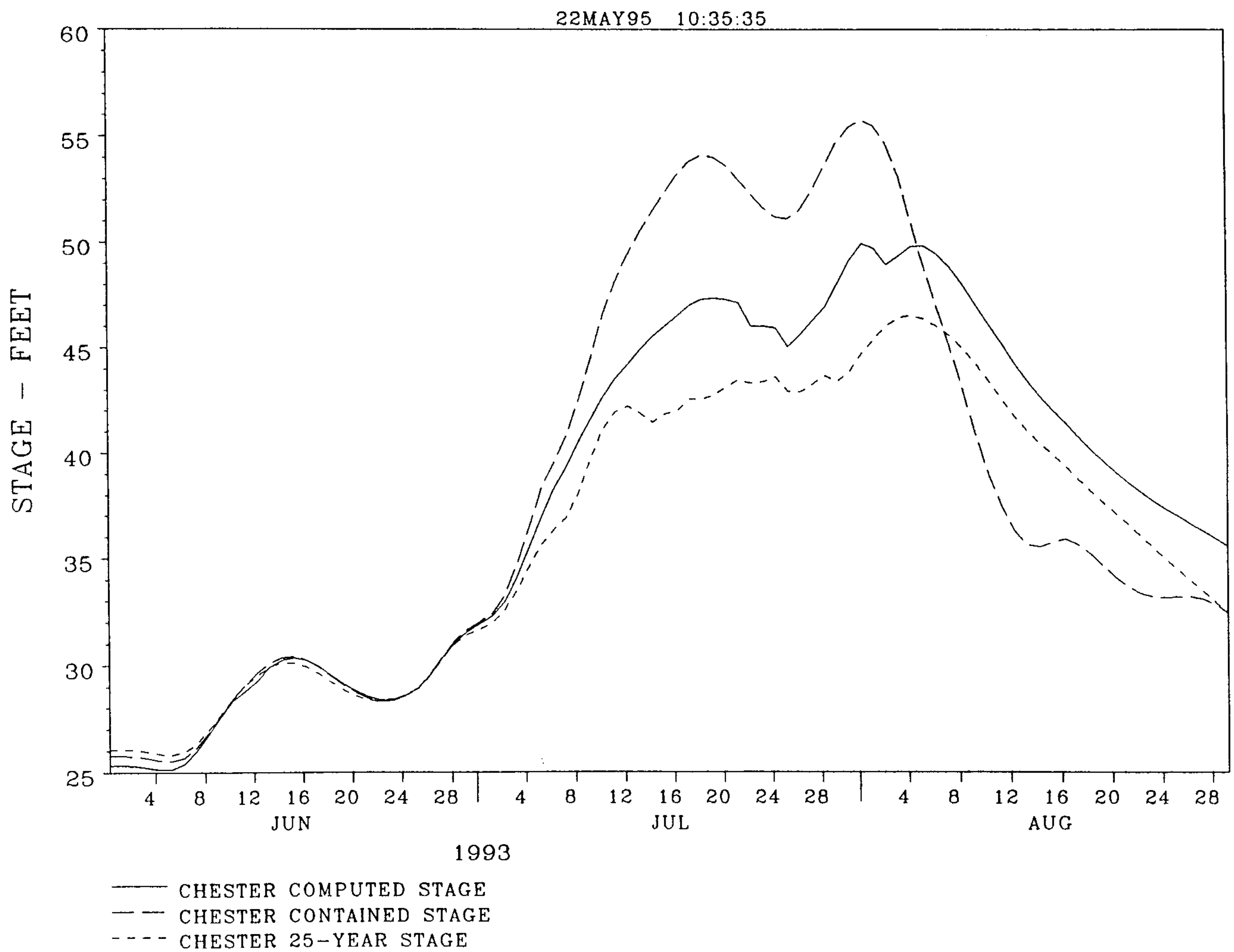


PLATE SL-10.3

MISSISSIPPI RIVER
CHESTER, IL GAGE – RM 109.9
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS

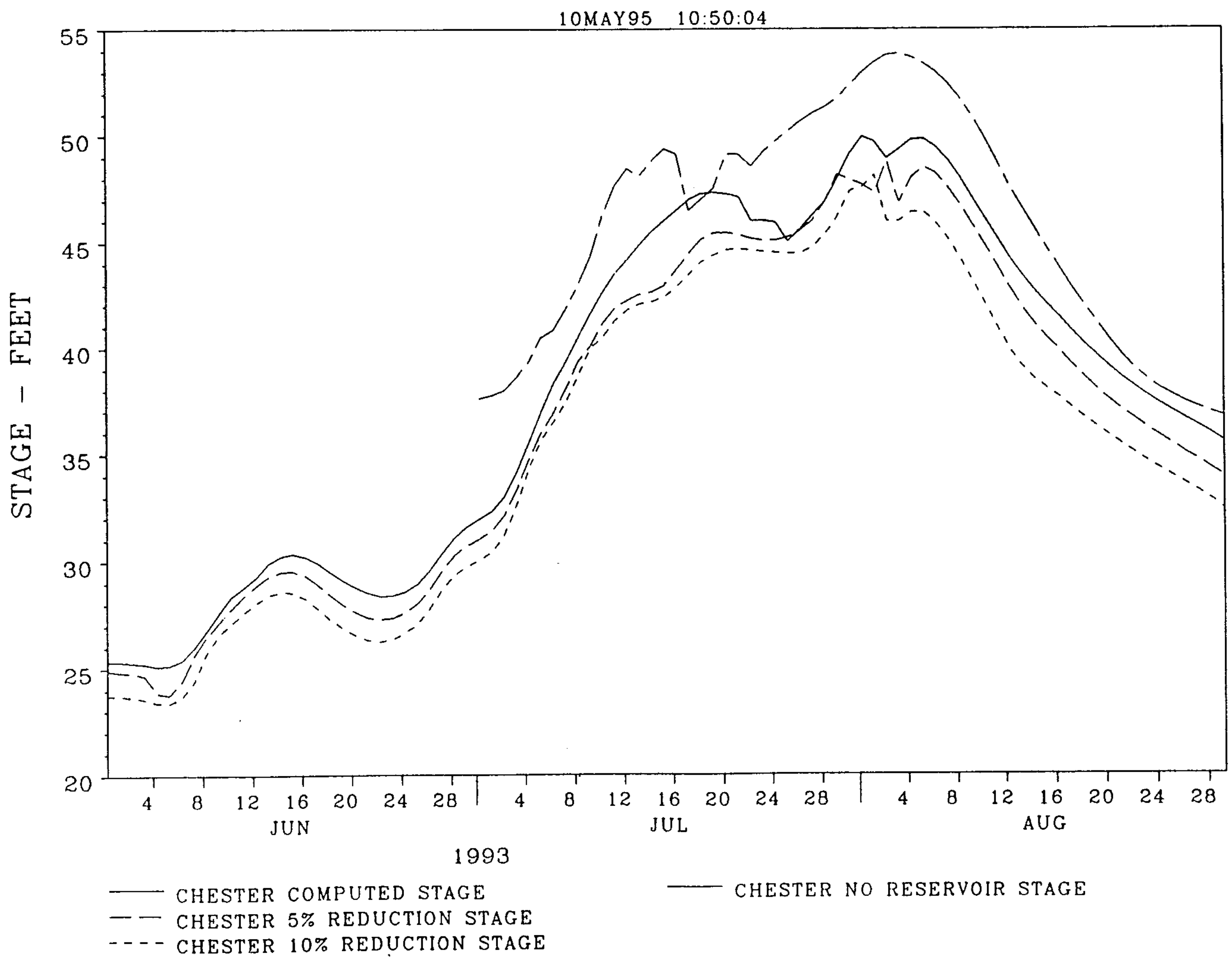


PLATE SL-10.4

MISSISSIPPI RIVER
CHESTER, IL GAGE – RM 109.9
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS

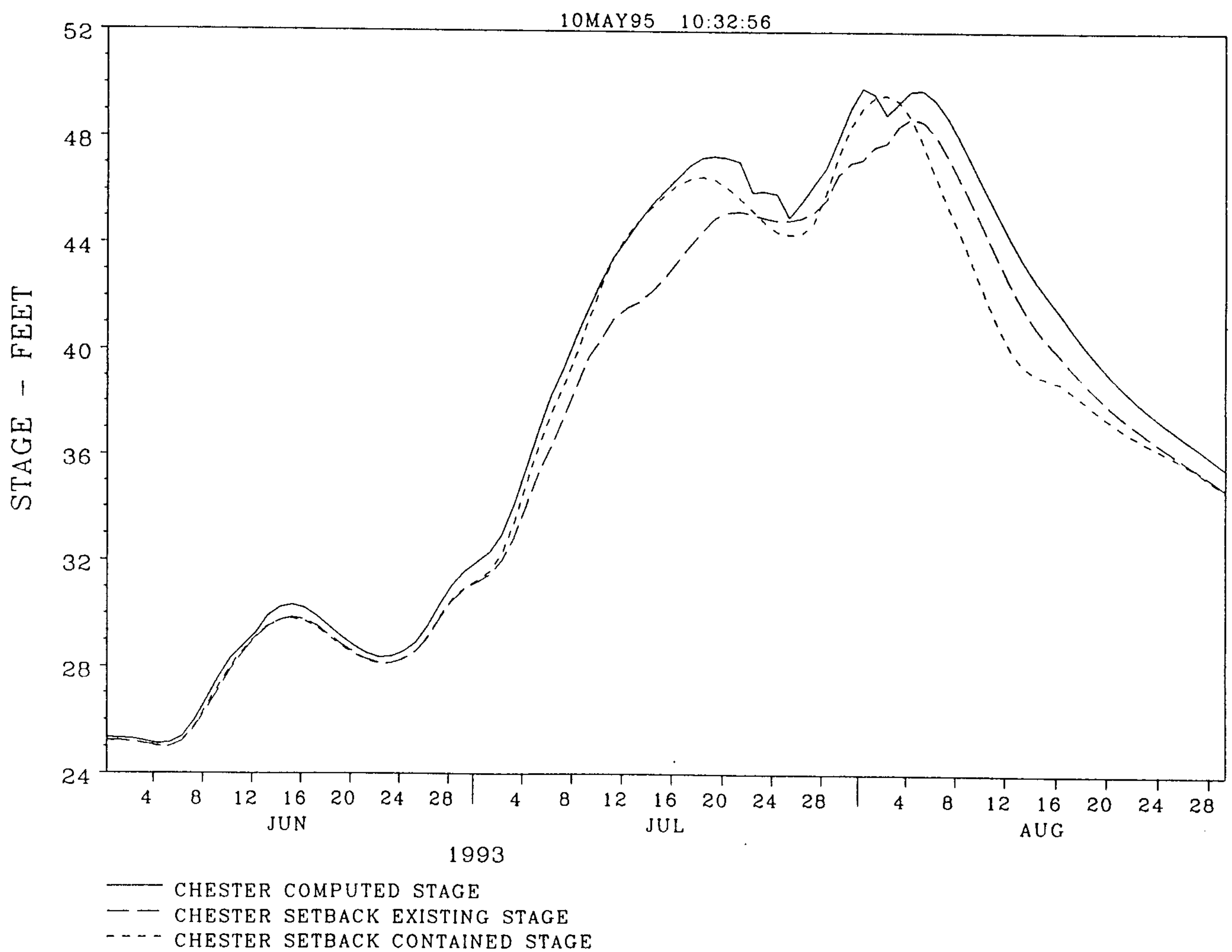


PLATE SL-10.5

MISSISSIPPI RIVER
GRAND TOWER, IL GAGE – RM 81.9
COMPUTED VS OBSERVED STAGES –1993 FLOOD

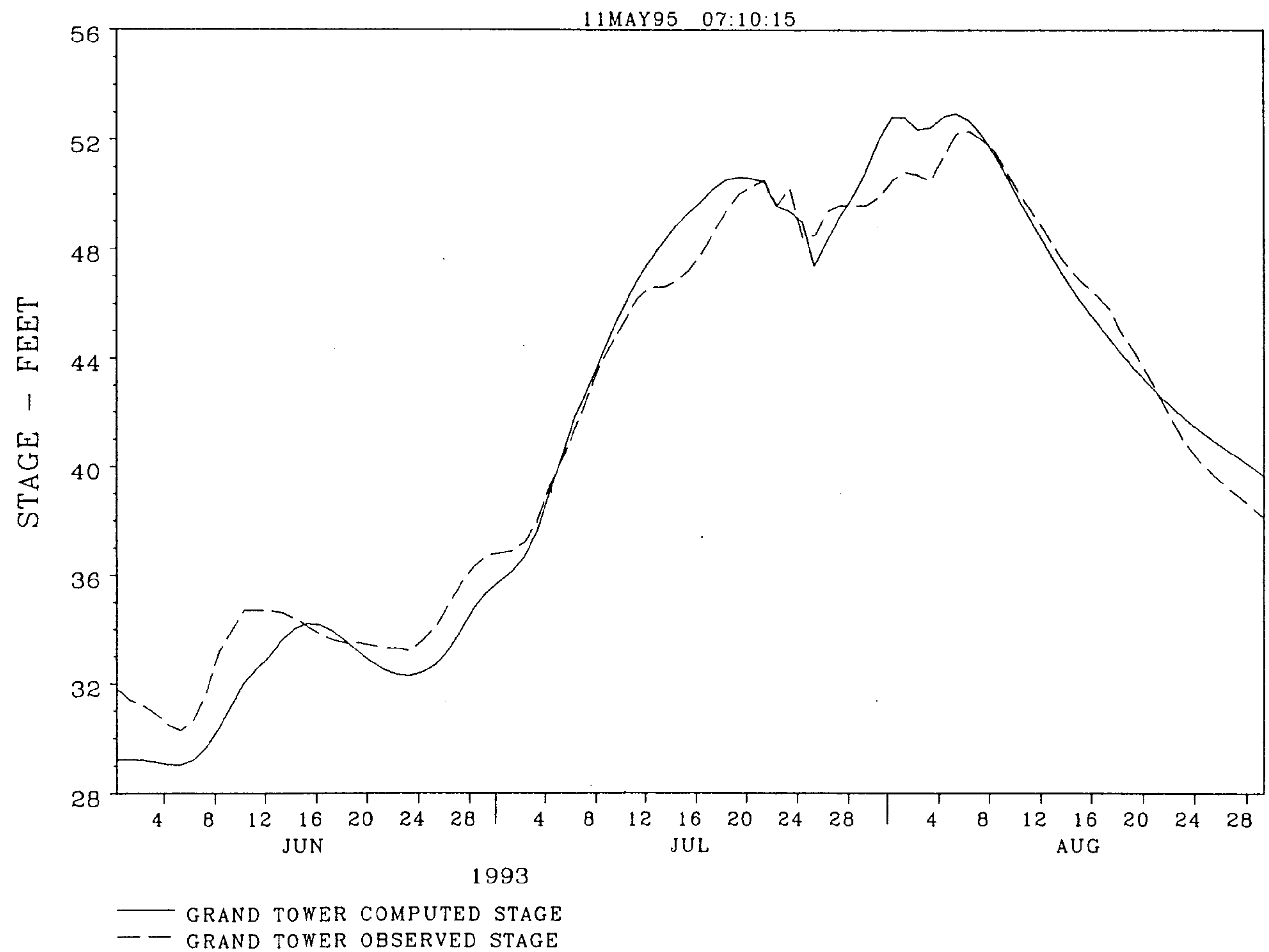
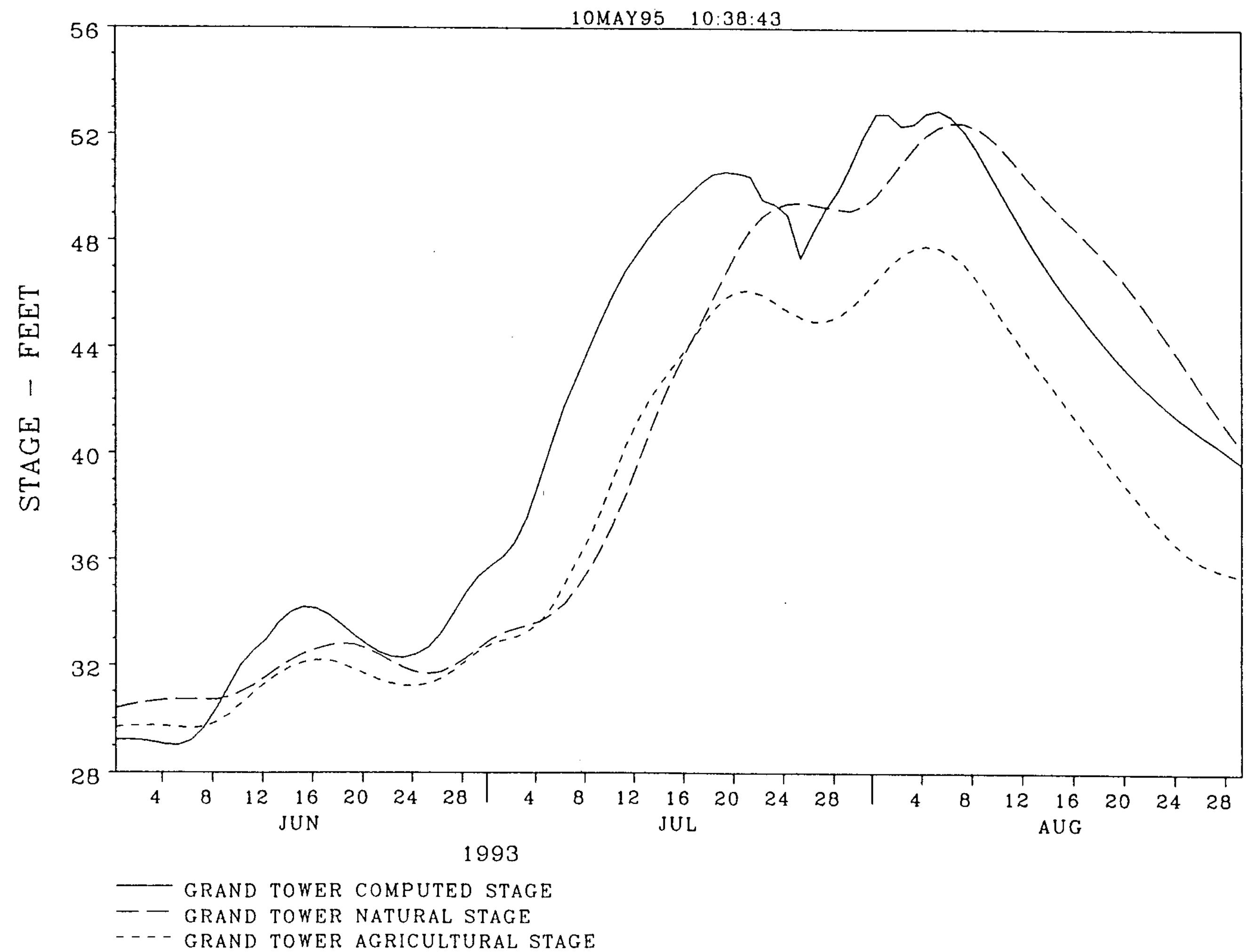
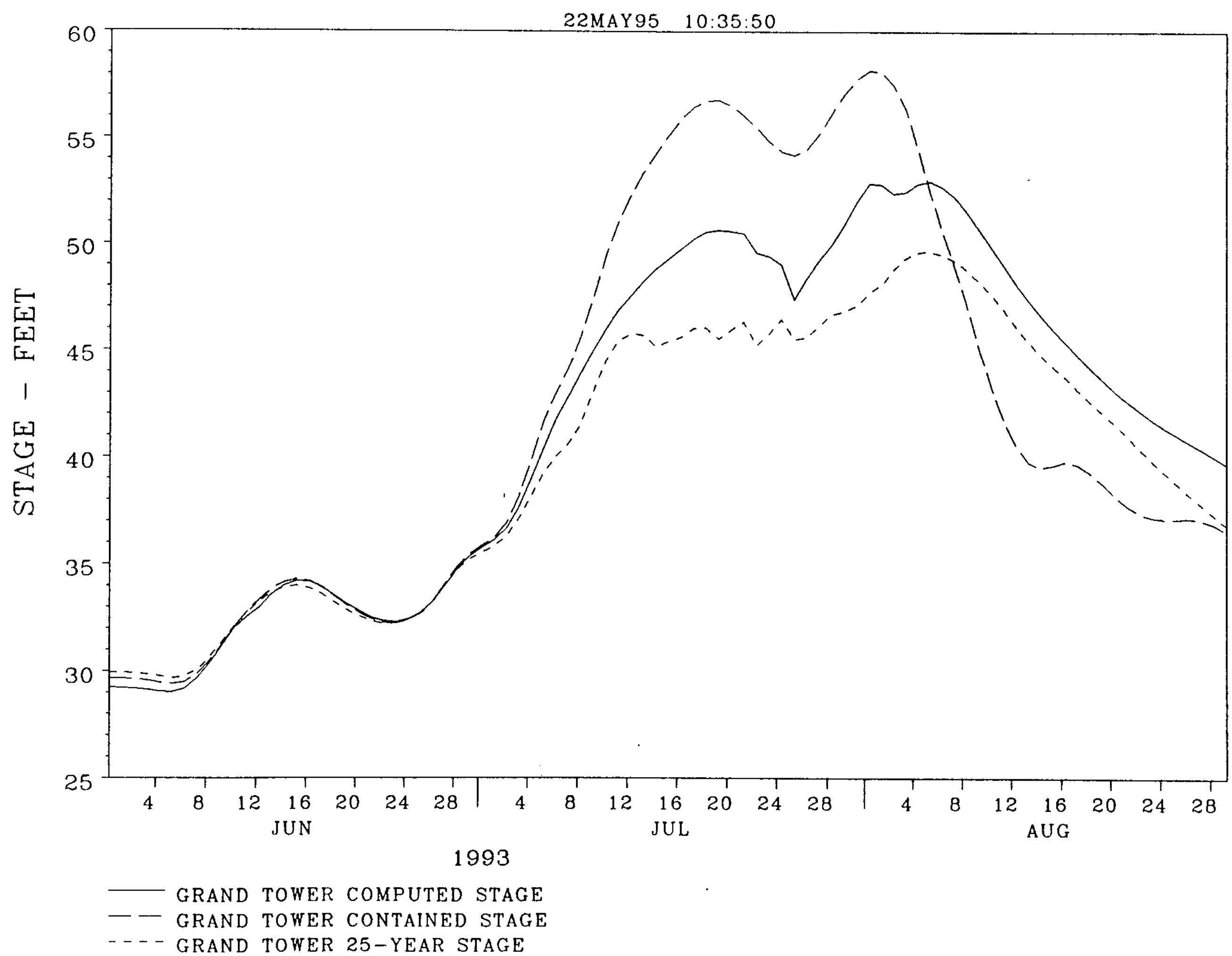


PLATE SL-11.1

MISSISSIPPI RIVER
GRAND TOWER, IL GAGE – RM 81.9
LEVEES REMOVED: AGRICULTUAL OR NATURAL OVERBANKS



MISSISSIPPI RIVER
GRAND TOWER, IL GAGE - RM 81.9
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



MISSISSIPPI RIVER
GRAND TOWER, IL GAGE – RM 81.9
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS

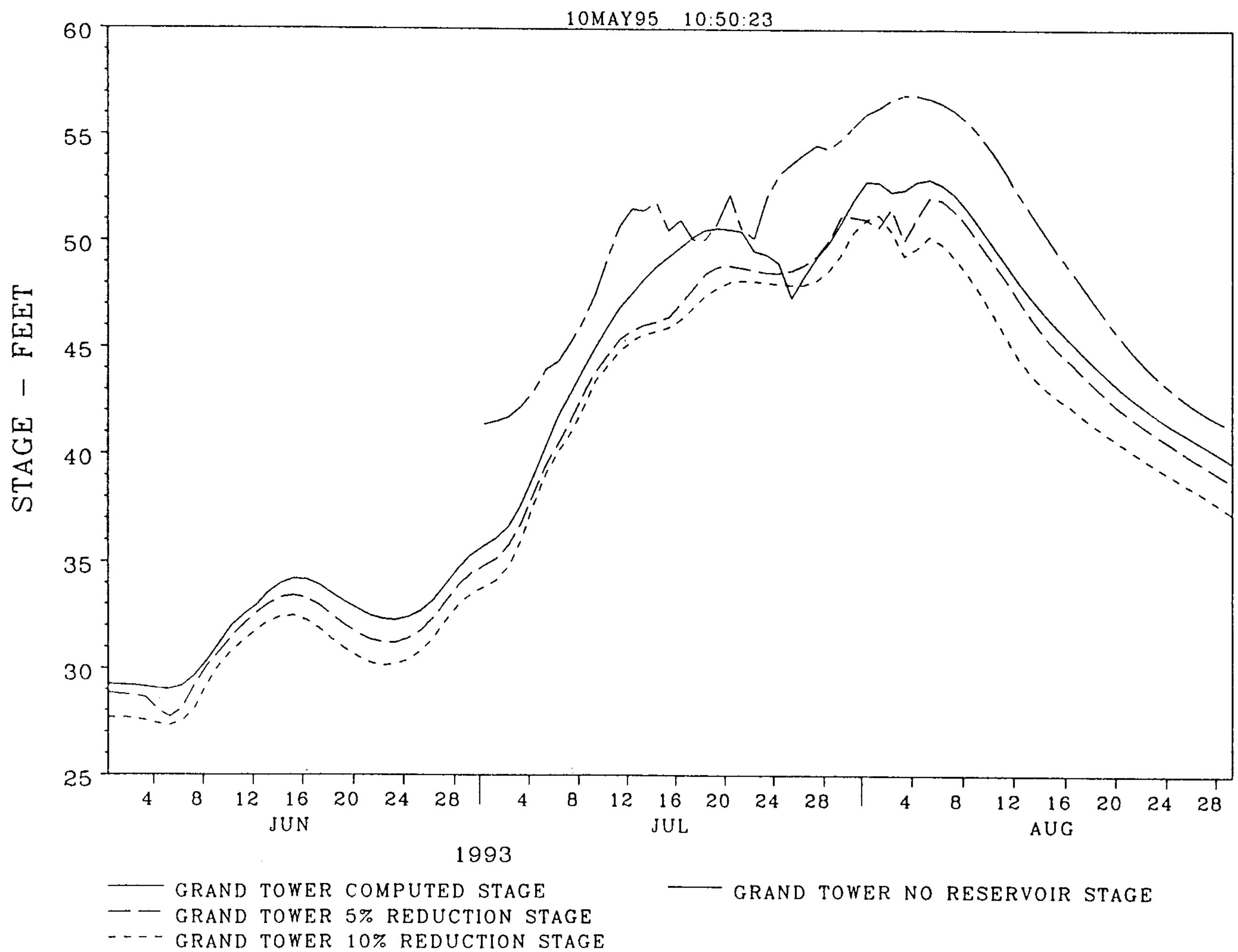


PLATE SL-11.4

MISSISSIPPI RIVER
GRAND TOWER, IL GAGE – RM 81.9
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS

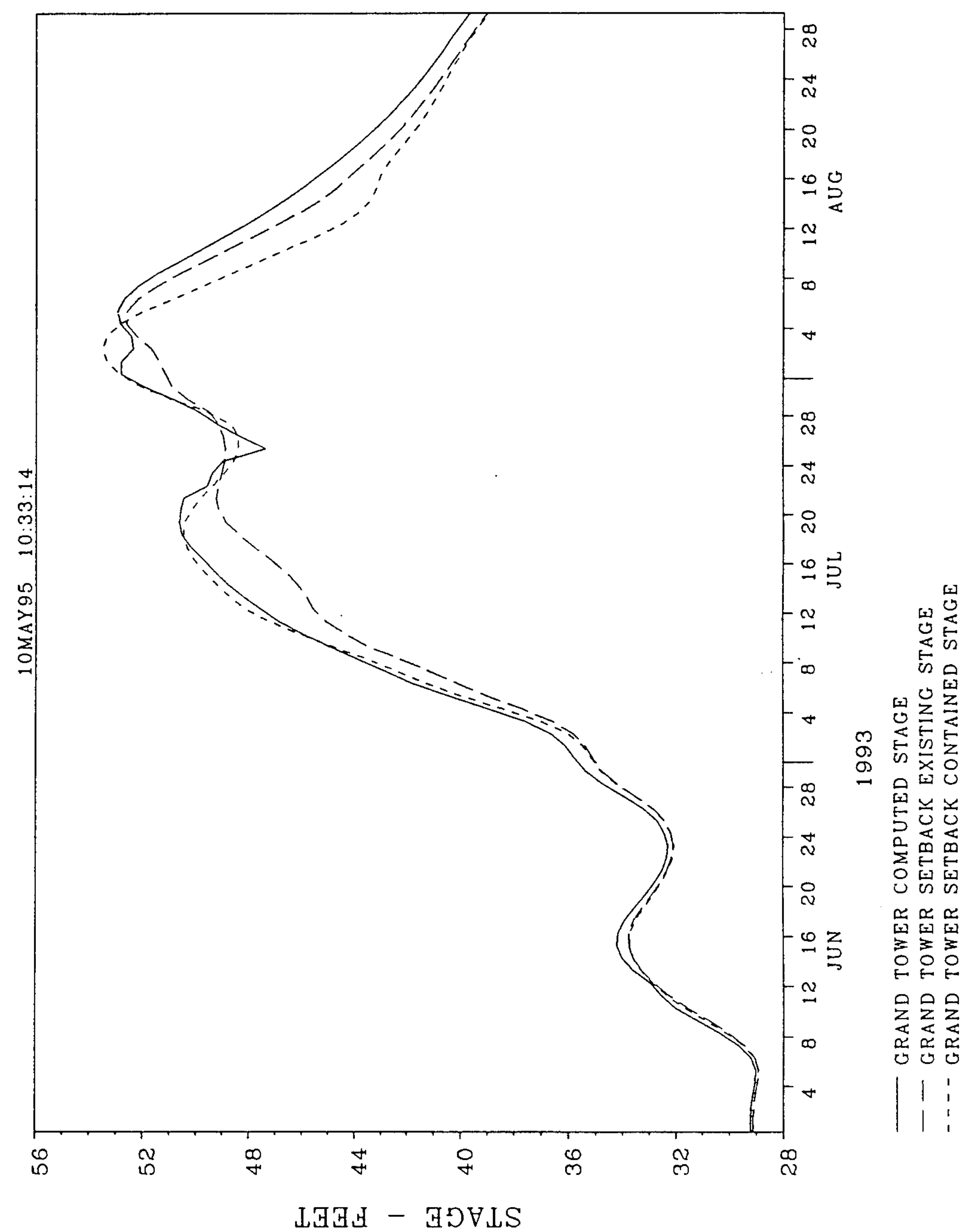


PLATE SL-11.5

MISSISSIPPI RIVER
CAPE GIRARDEAU, MO GAGE – RM 52.0
COMPUTED VS OBSERVED STAGES –1993 FLOOD

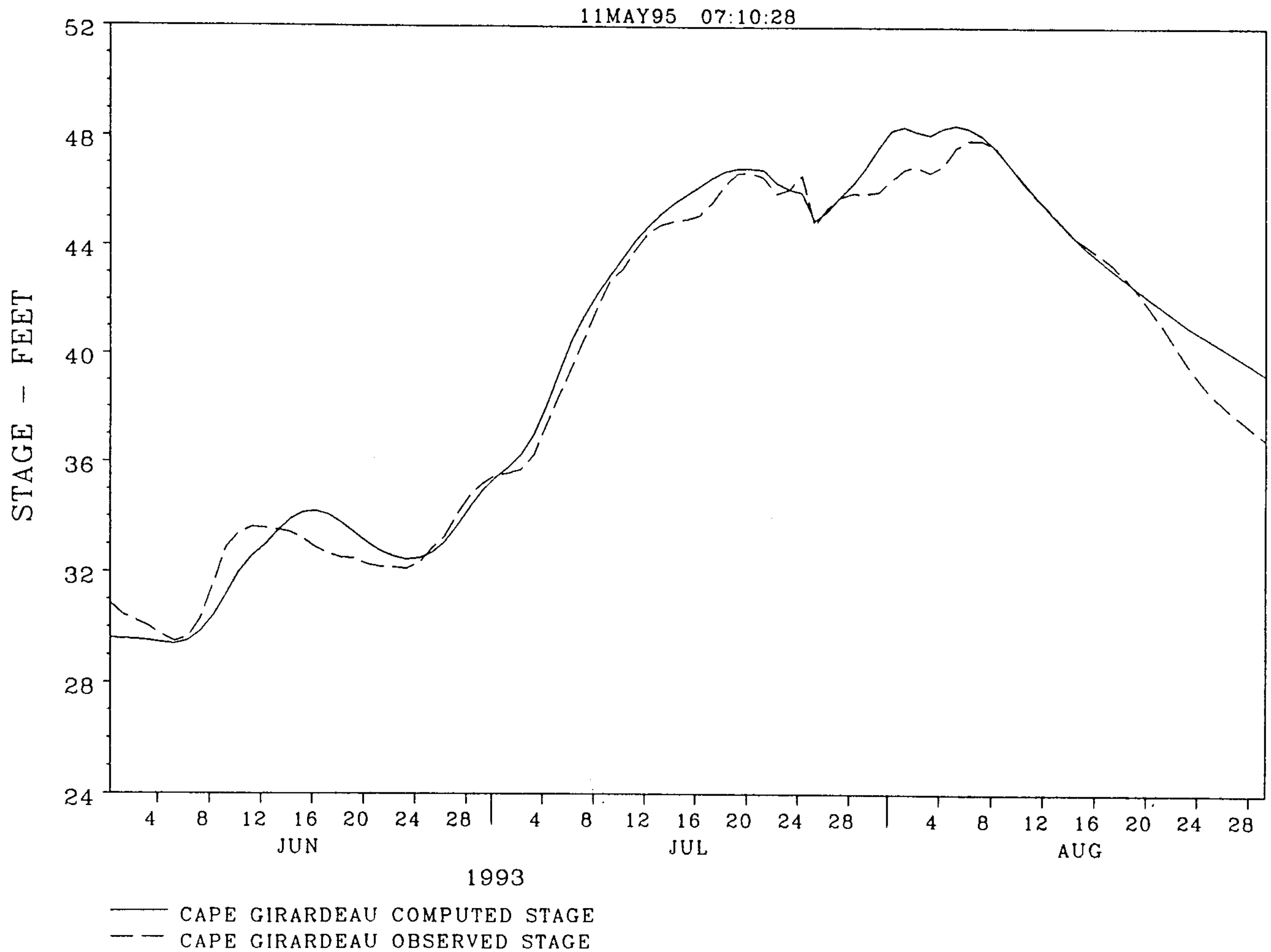


PLATE SL-12.1

MISSISSIPPI RIVER
CAPE GIRARDEAU, MO GAGE – RM 52.0
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

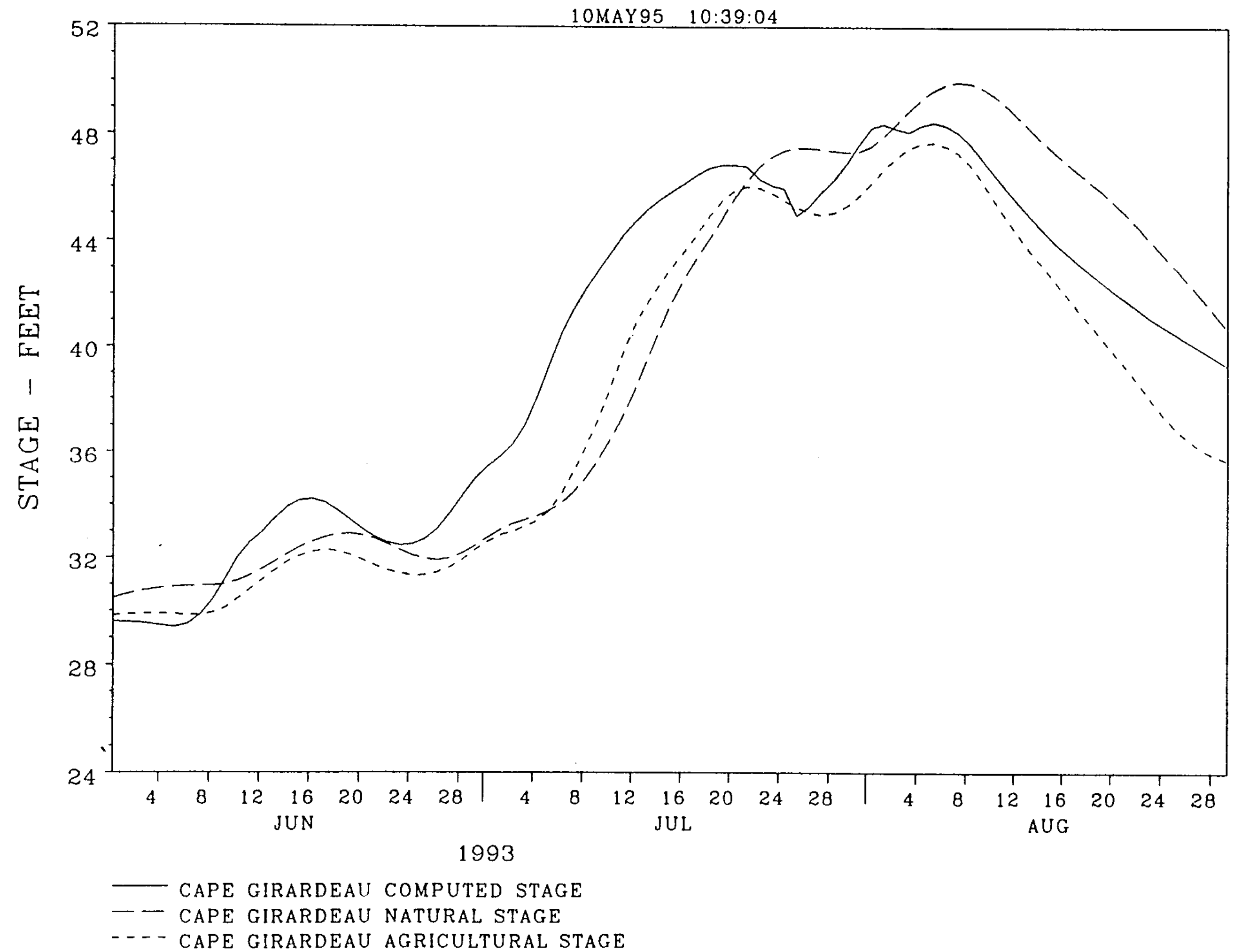


PLATE SL-12.2

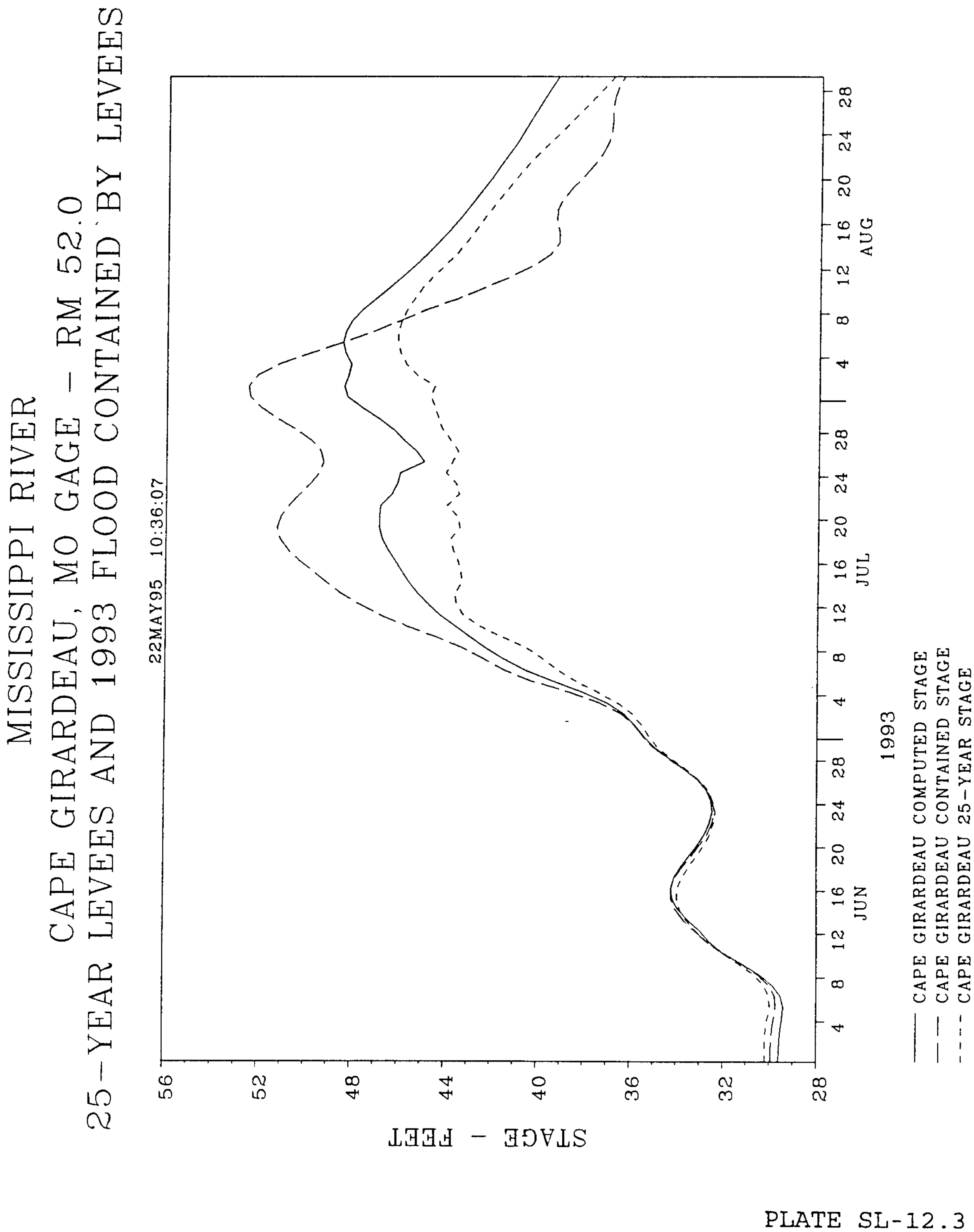

MISSISSIPPI RIVER
CAPE GIRARDEAU, MO GAGE - RM 52.0
25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES
22MAY95 10:36:07
STAGE - FEET
28
32
36
40
44
48
52
56
4 8 12 16 20 24 28
JUN
4 8 12 16 20 24 28
JUL
4 8 12 16 20 24 28
AUG
1993
CAPE GIRARDEAU COMPUTED STAGE
CAPE GIRARDEAU CONTAINED STAGE
CAPE GIRARDEAU 25-YEAR STAGE

PLATE SL-12.3

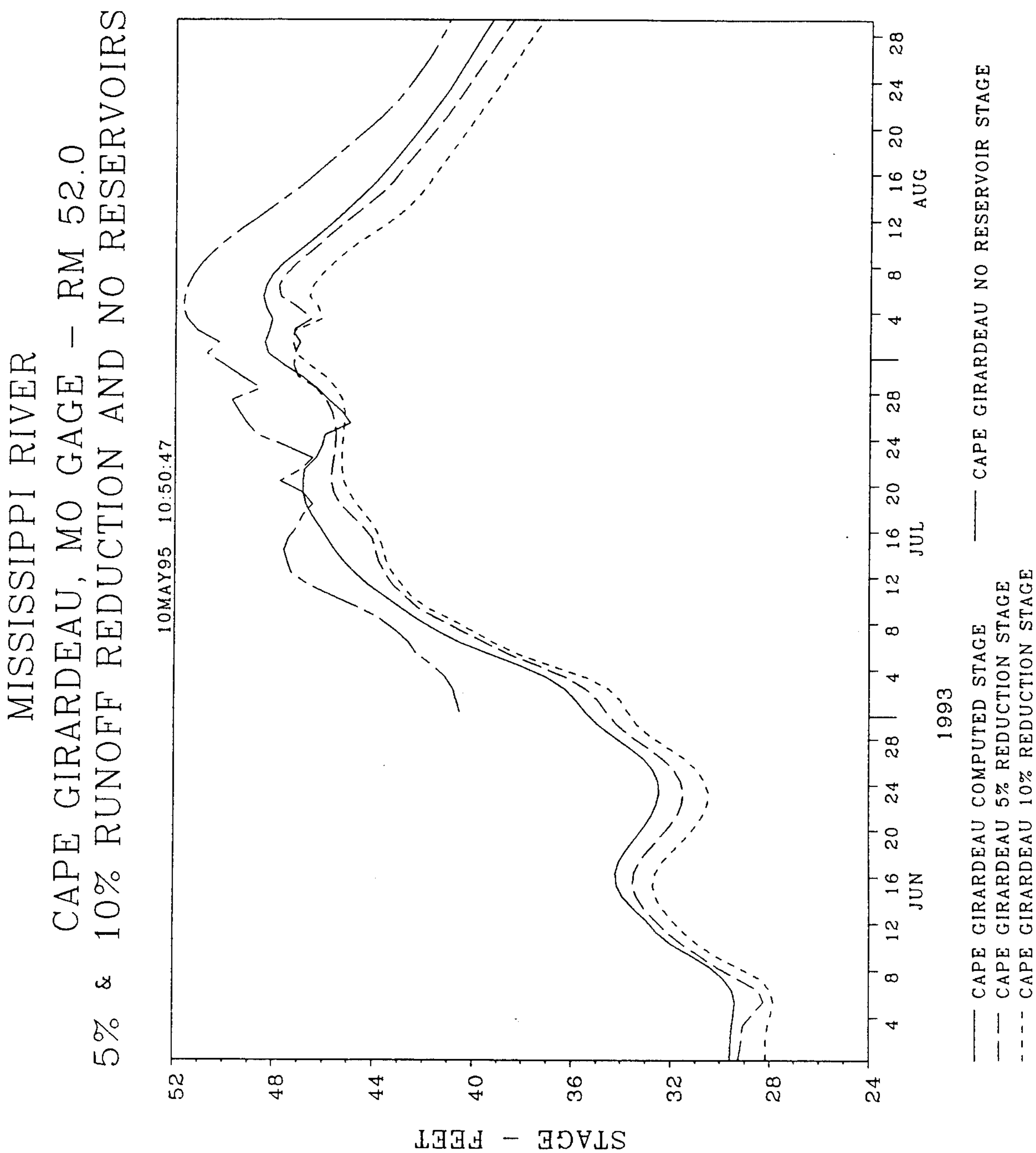

MISSISSIPPI RIVER
CAPE GIRARDEAU, MO GAGE – RM 52.0
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS
10MAY95 10:50:47
STAGE – FEET
52
48
44
40
36
32
28
24
4 8 12 16 20 24 28
JUN
4 8 12 16 20 24 28
JUL
4 8 12 16 20 24 28
AUG
1993
CAPE GIRARDEAU COMPUTED STAGE
CAPE GIRARDEAU 5% REDUCTION STAGE
CAPE GIRARDEAU 10% REDUCTION STAGE
CAPE GIRARDEAU NO RESERVOIR STAGE

PLATE SL-12.4

MISSISSIPPI RIVER
CAPE GIRARDEAU, MO GAGE – RM 52.0
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS

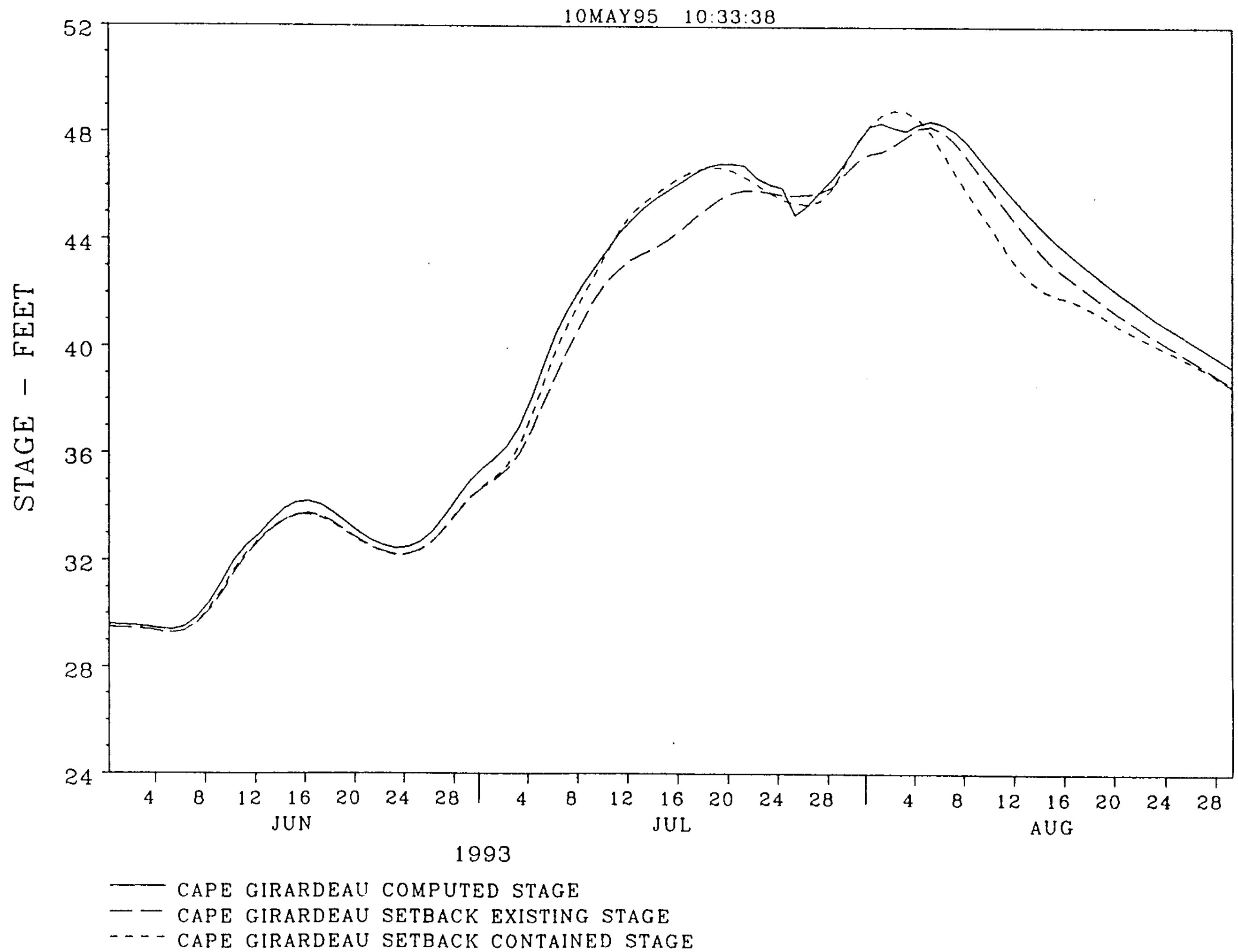


PLATE SL-12.5

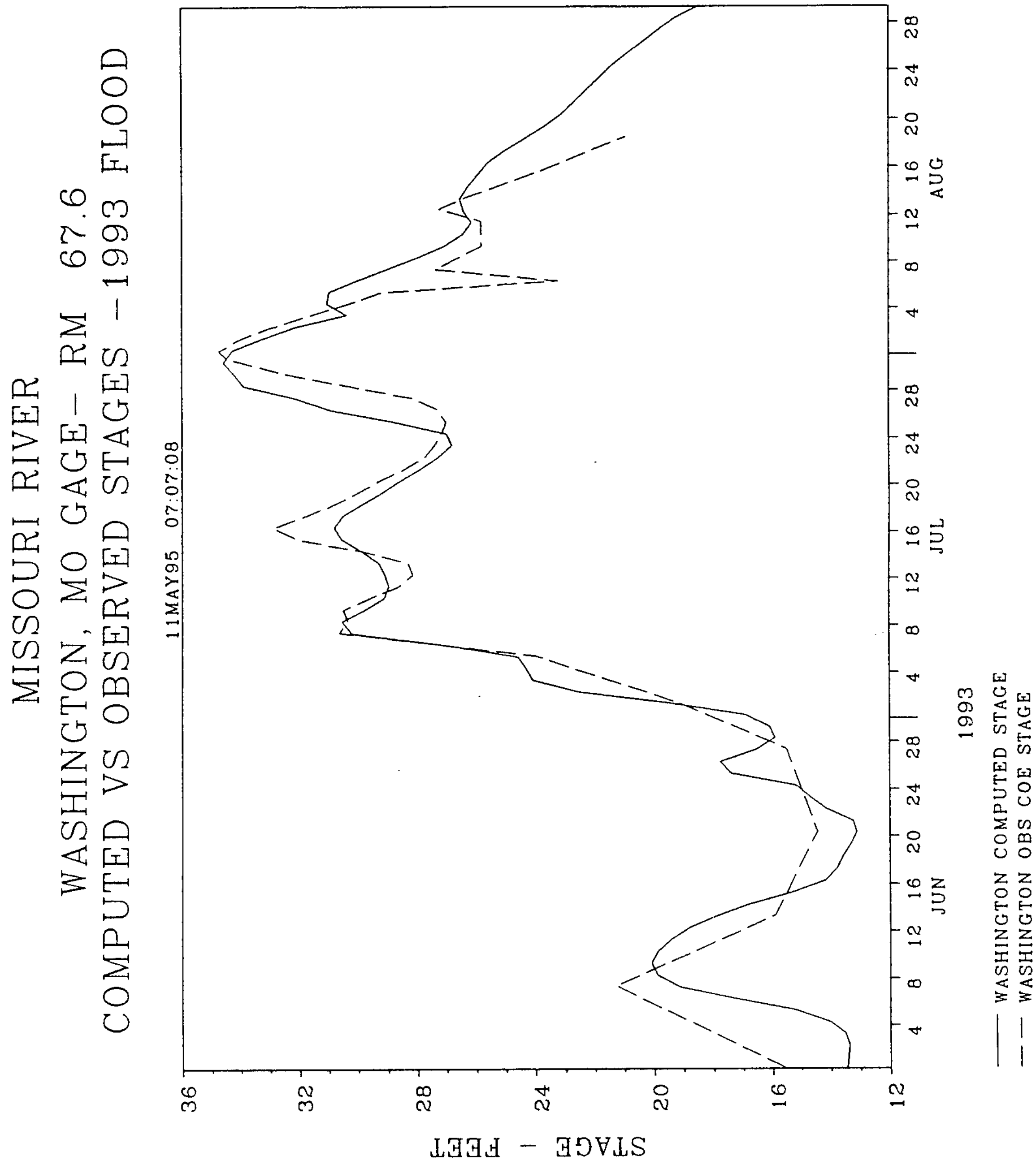

MISSOURI RIVER
WASHINGTON, MO GAGE- RM 67.6
COMPUTED VS OBSERVED STAGES -1993 FLOOD
11MAY95 07:07:08
STAGE - FEET
36
32
28
24
20
16
12
4
8
12
16
20
24
28
JUN
4
8
12
16
20
24
28
JUL
4
8
12
16
20
24
28
AUG
1993
WASHINGTON COMPUTED STAGE
WASHINGTON OBS COE STAGE

PLATE SL-13.1

MISSOURI RIVER
WASHINGTON, MO GAGE– RM 67.6
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

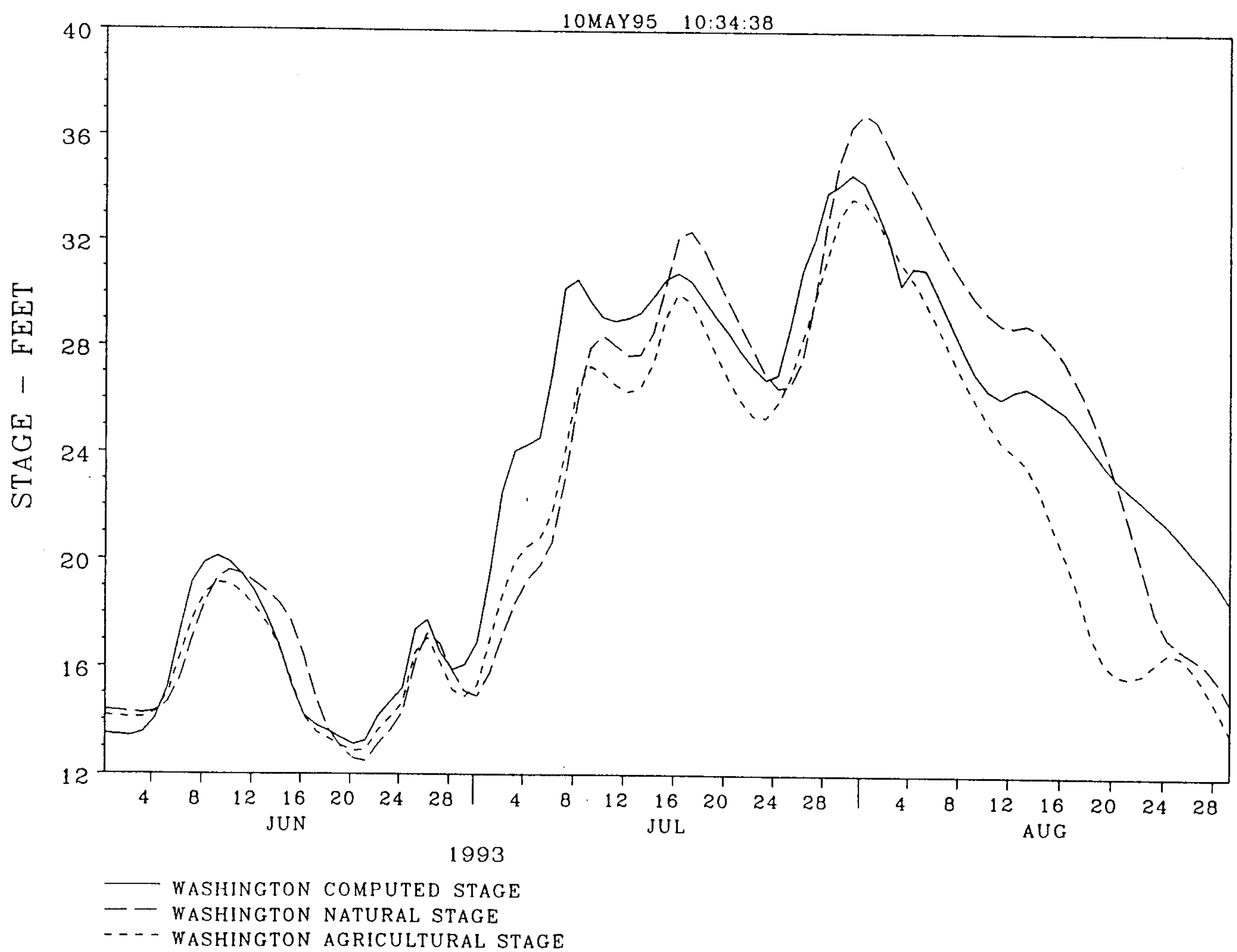
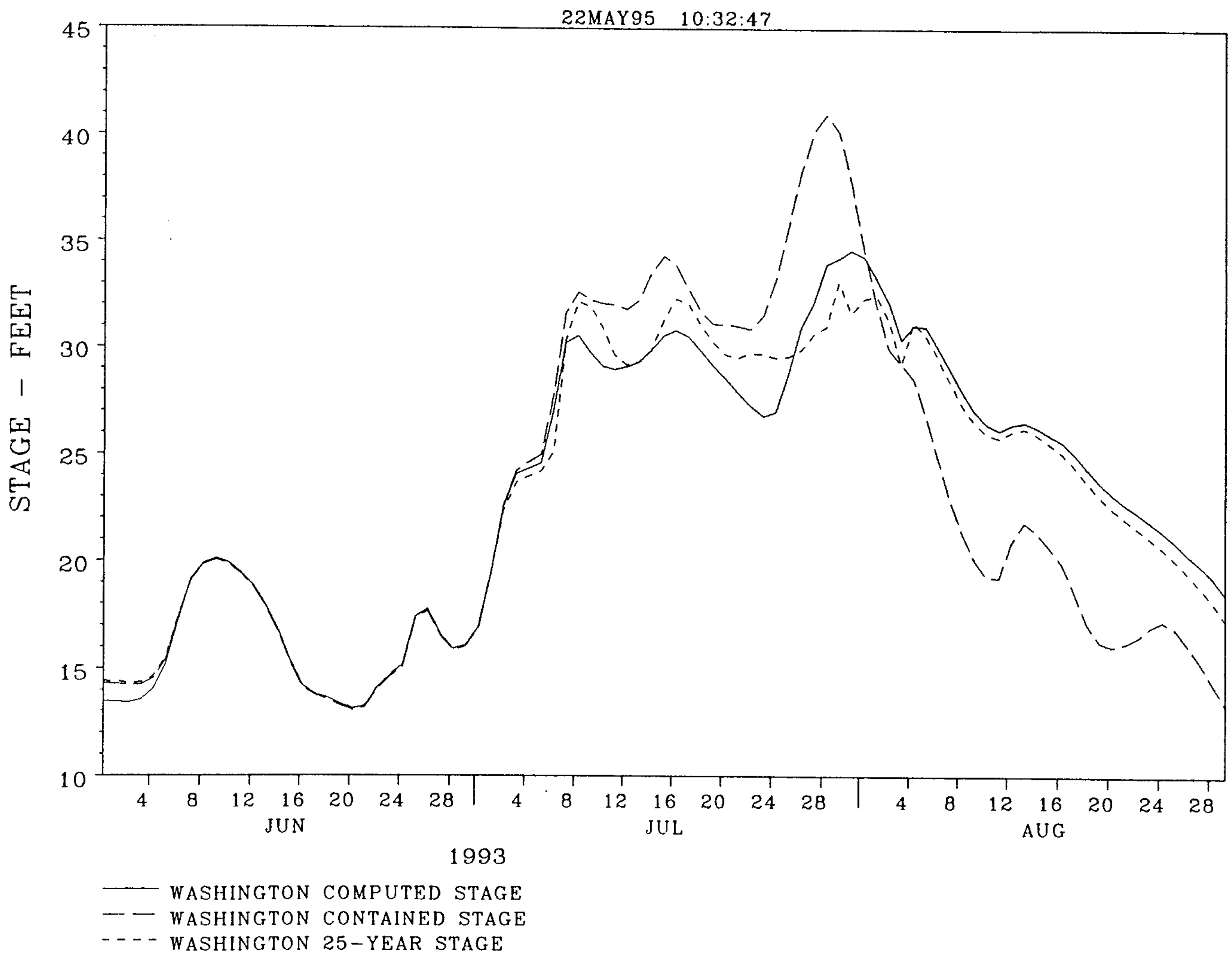
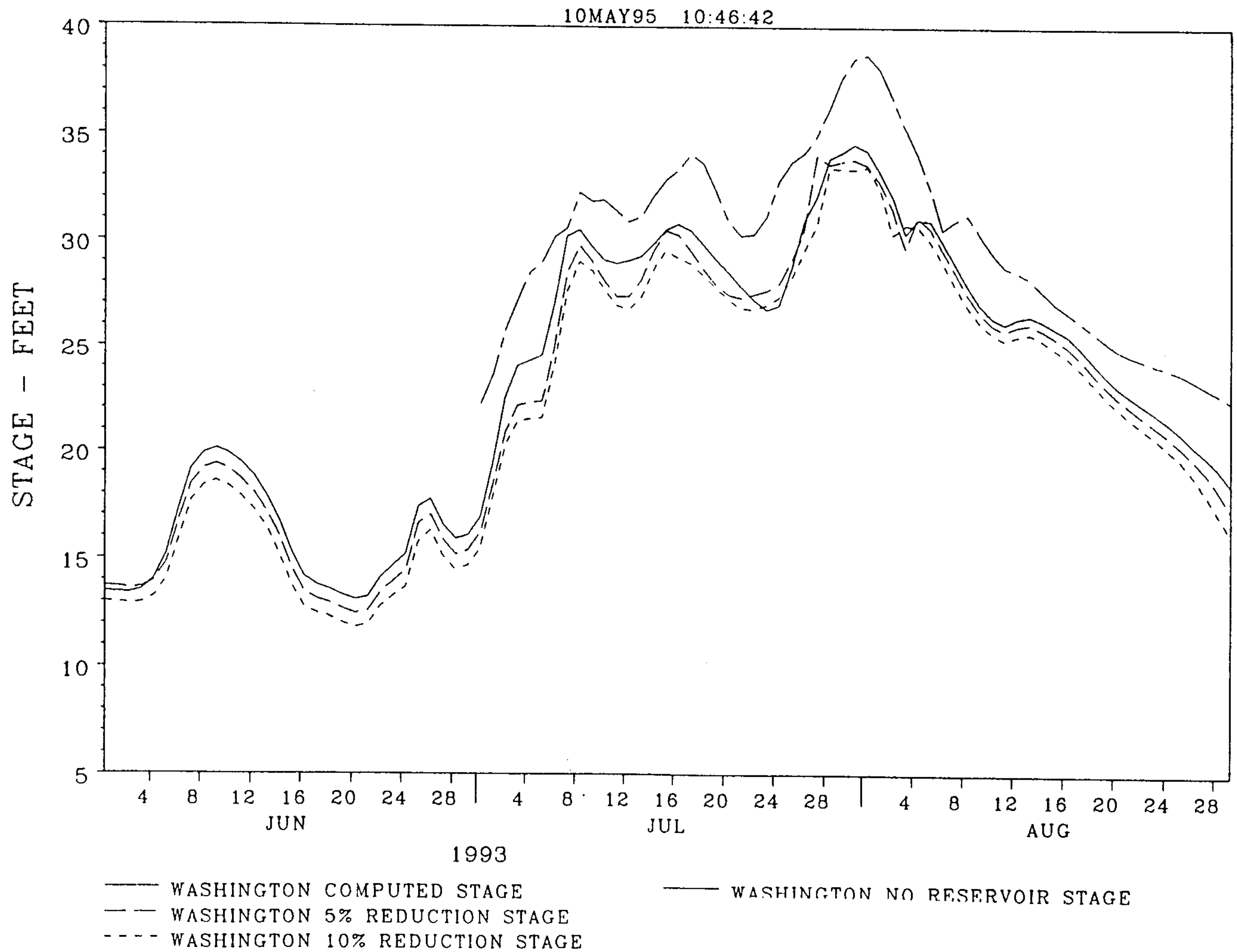


PLATE SL-13.2

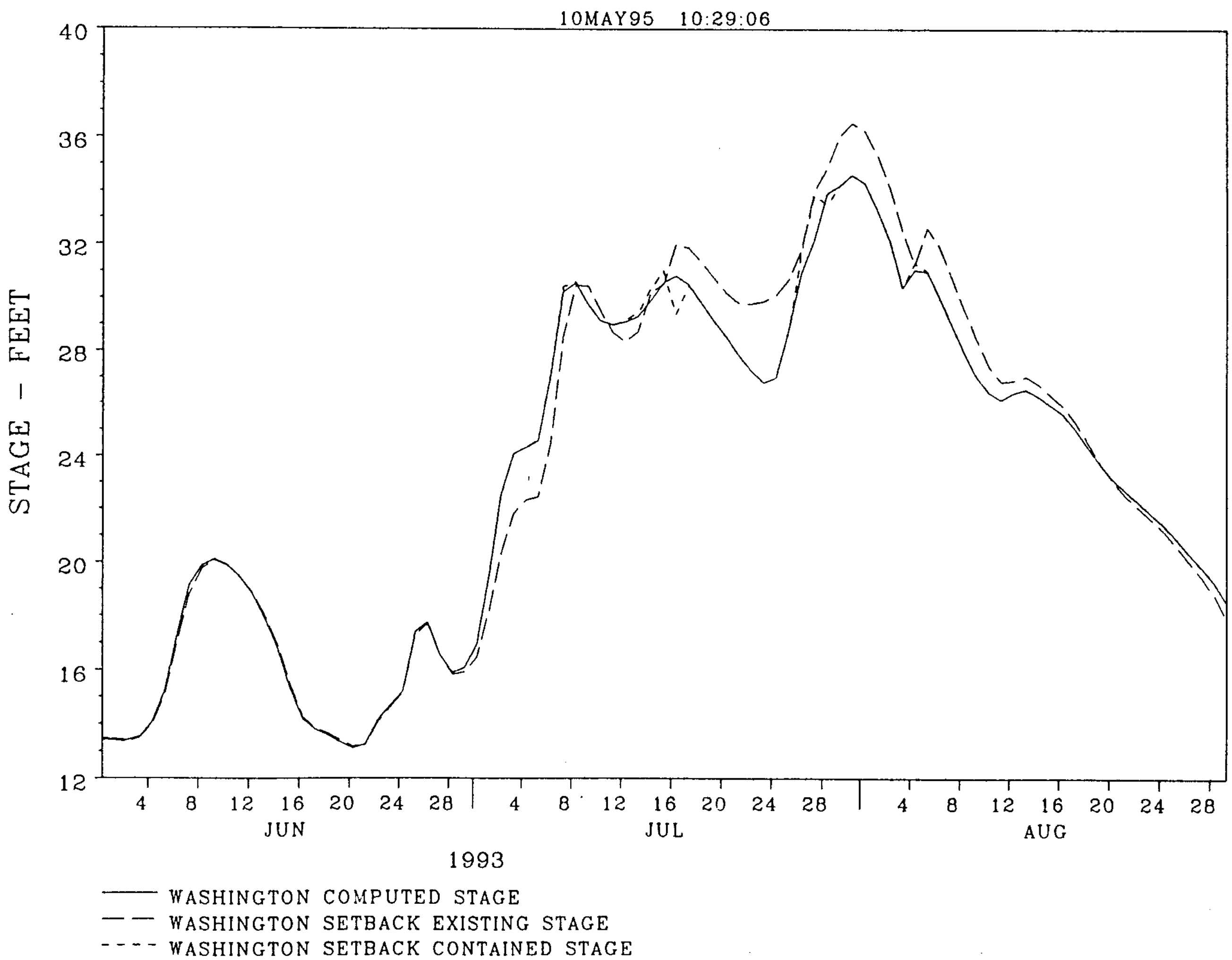
MISSOURI RIVER
WASHINGTON, MO GAGE– RM 67.6
25–YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



MISSOURI RIVER
WASHINGTON, MO GAGE– RM 67.6
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS



MISSOURI RIVER
WASHINGTON, MO GAGE – RM 67.6
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS



MISSOURI RIVER
ST. CHARLES, MO GAGE - RM 28.2
COMPUTED VS OBSERVED STAGES -1993 FLOOD

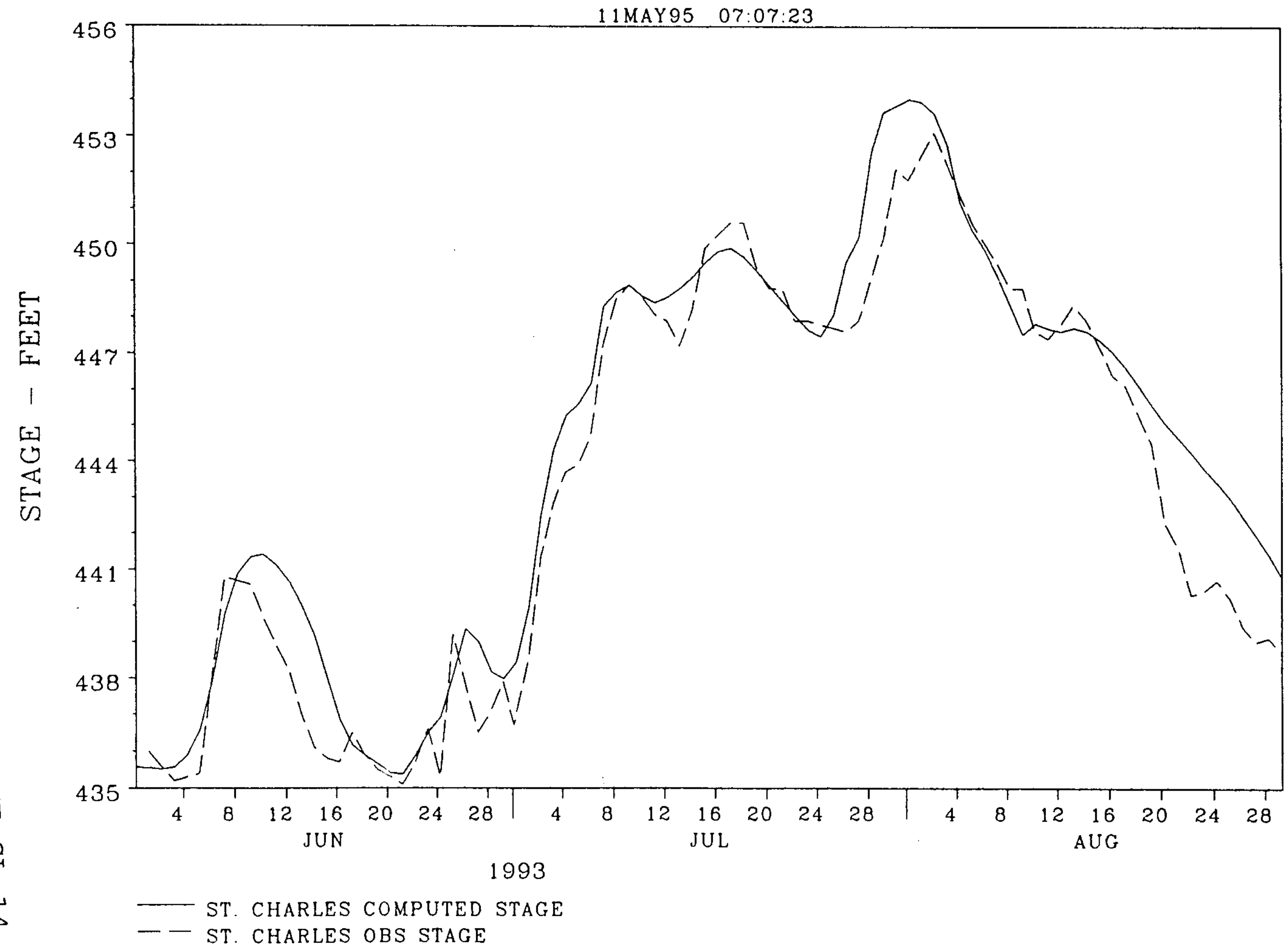


PLATE SL-14.1

MISSOURI RIVER
ST. CHARLES, MO GAGE – RM 28.2
LEVEES REMOVED: AGRICULTURAL OR NATURAL OVERBANKS

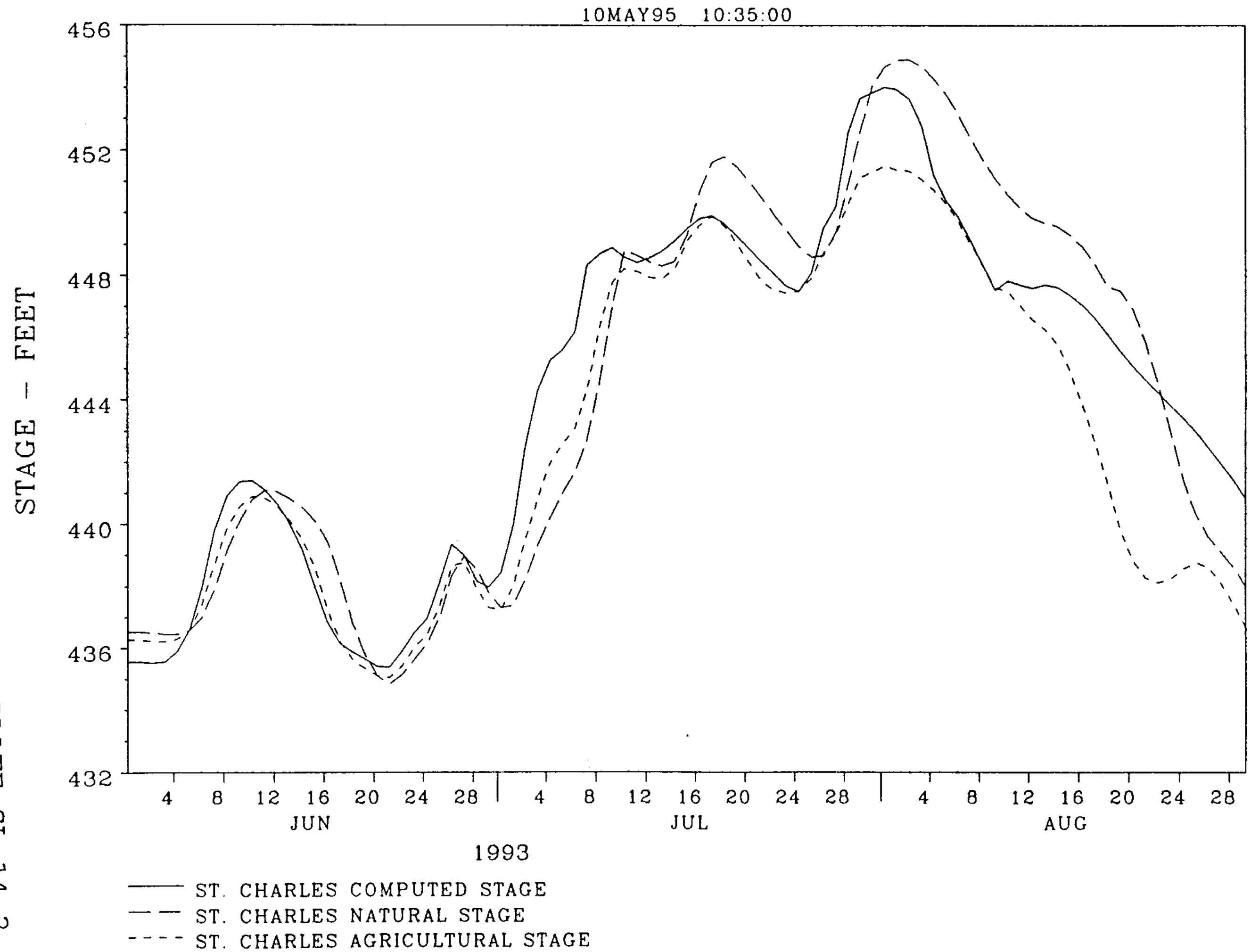
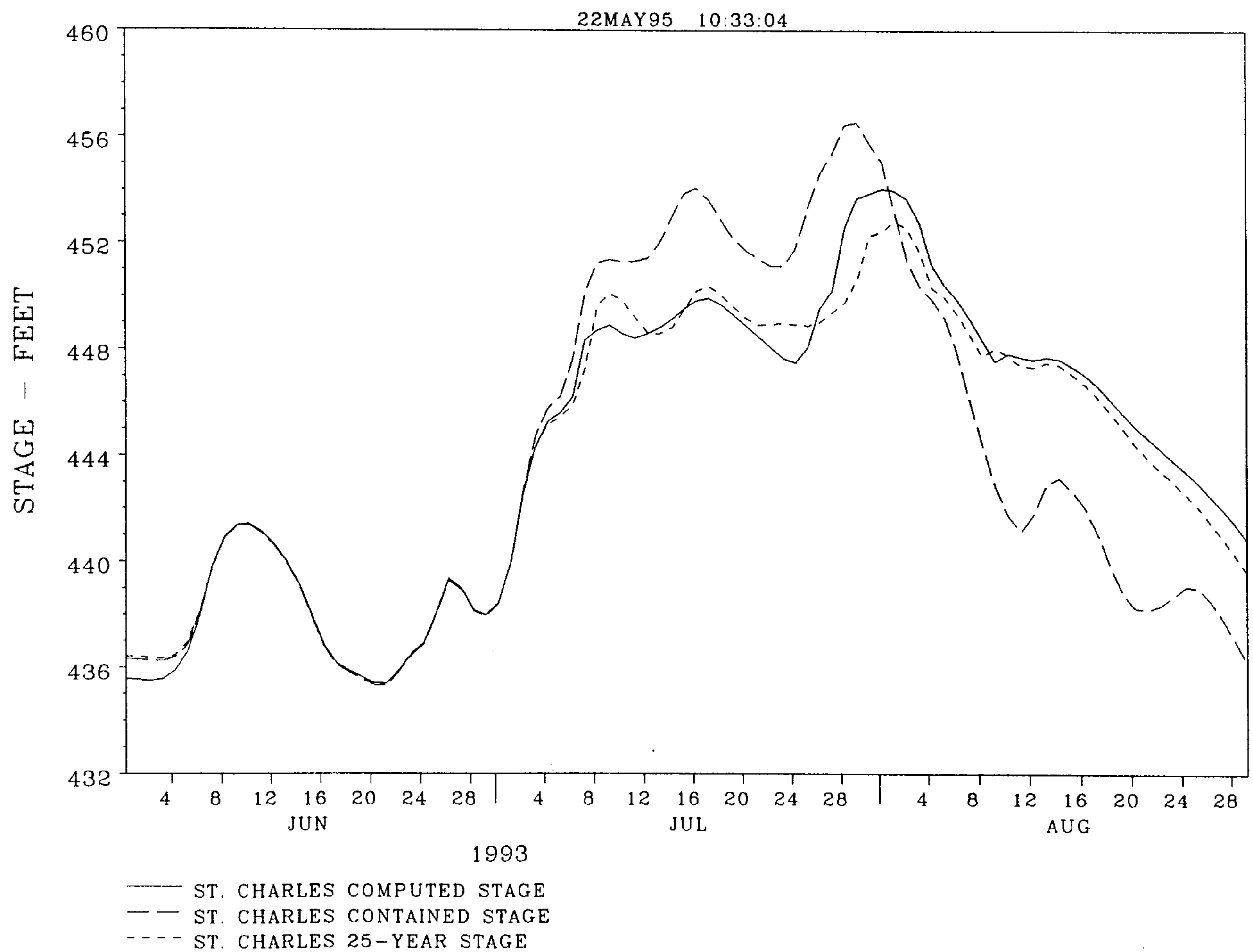


PLATE SL-14.2

MISSOURI RIVER

ST. CHARLES, MO GAGE – RM 28.2

25-YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES



MISSOURI RIVER
ST. CHARLES, MO GAGE – RM 28.2
25–YEAR LEVEES AND 1993 FLOOD CONTAINED BY LEVEES

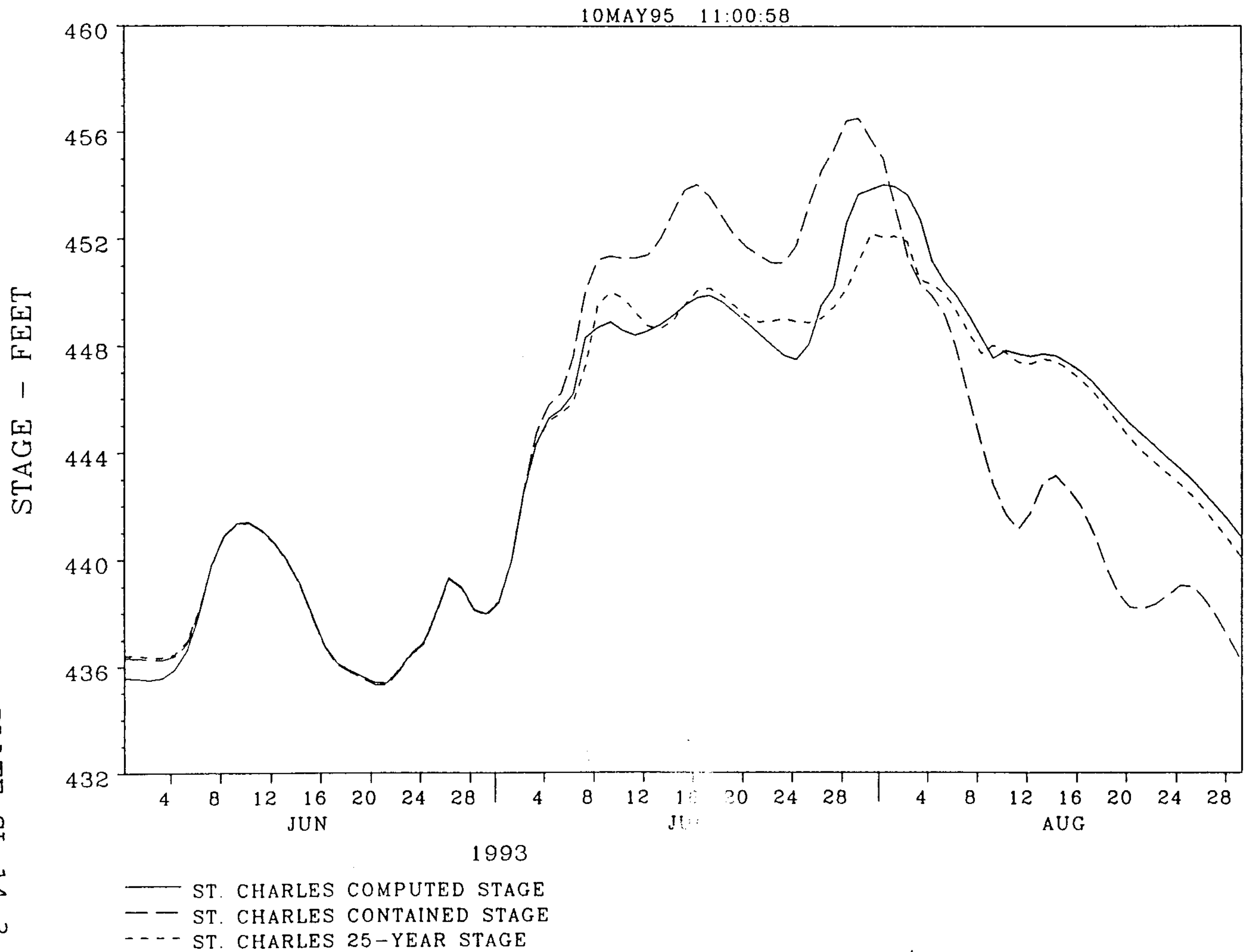


PLATE SL-14.3

MISSOURI RIVER
ST. CHARLES, MO GAGE - RM 28.2
5% & 10% RUNOFF REDUCTION AND NO RESERVOIRS

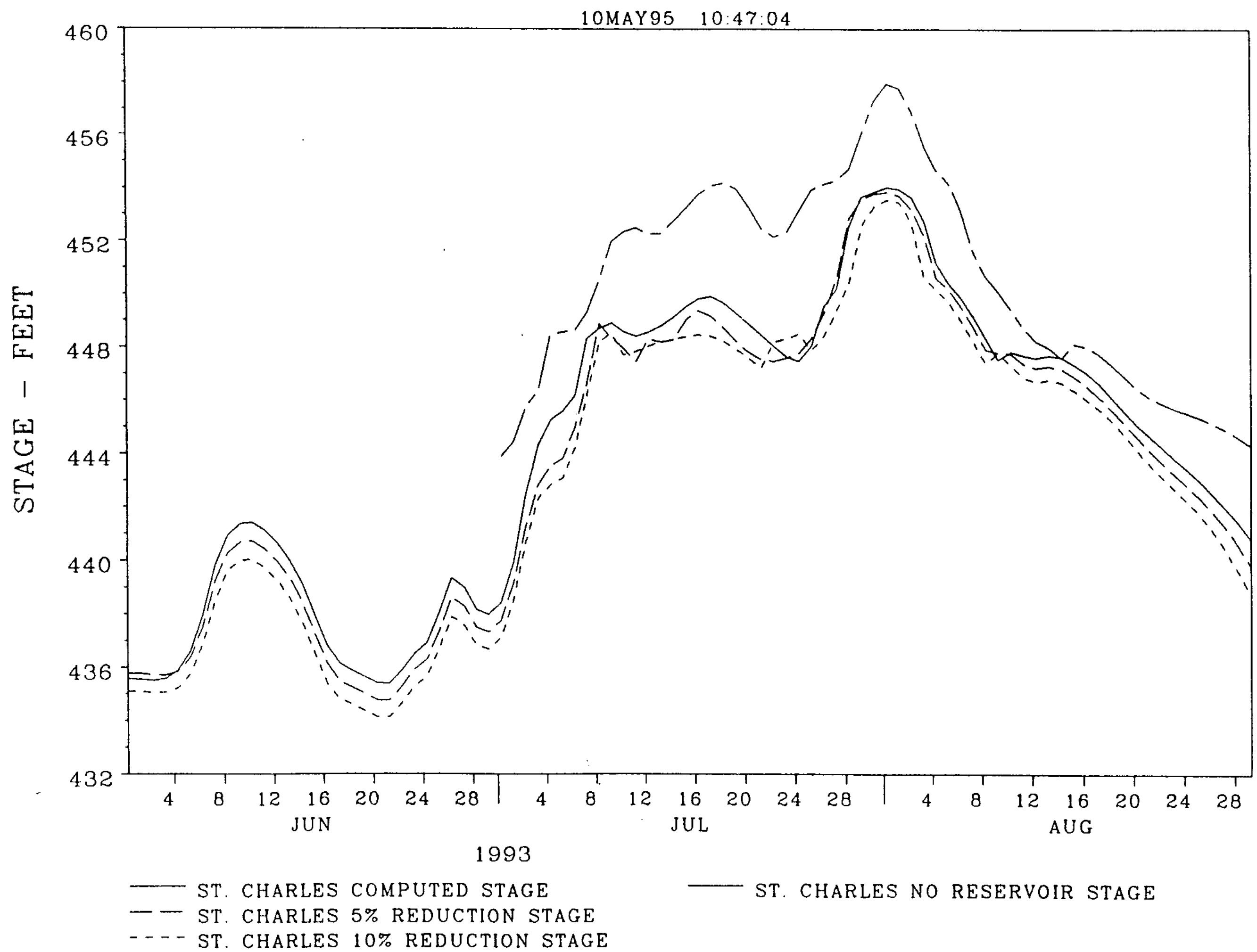


PLATE SL-14.4

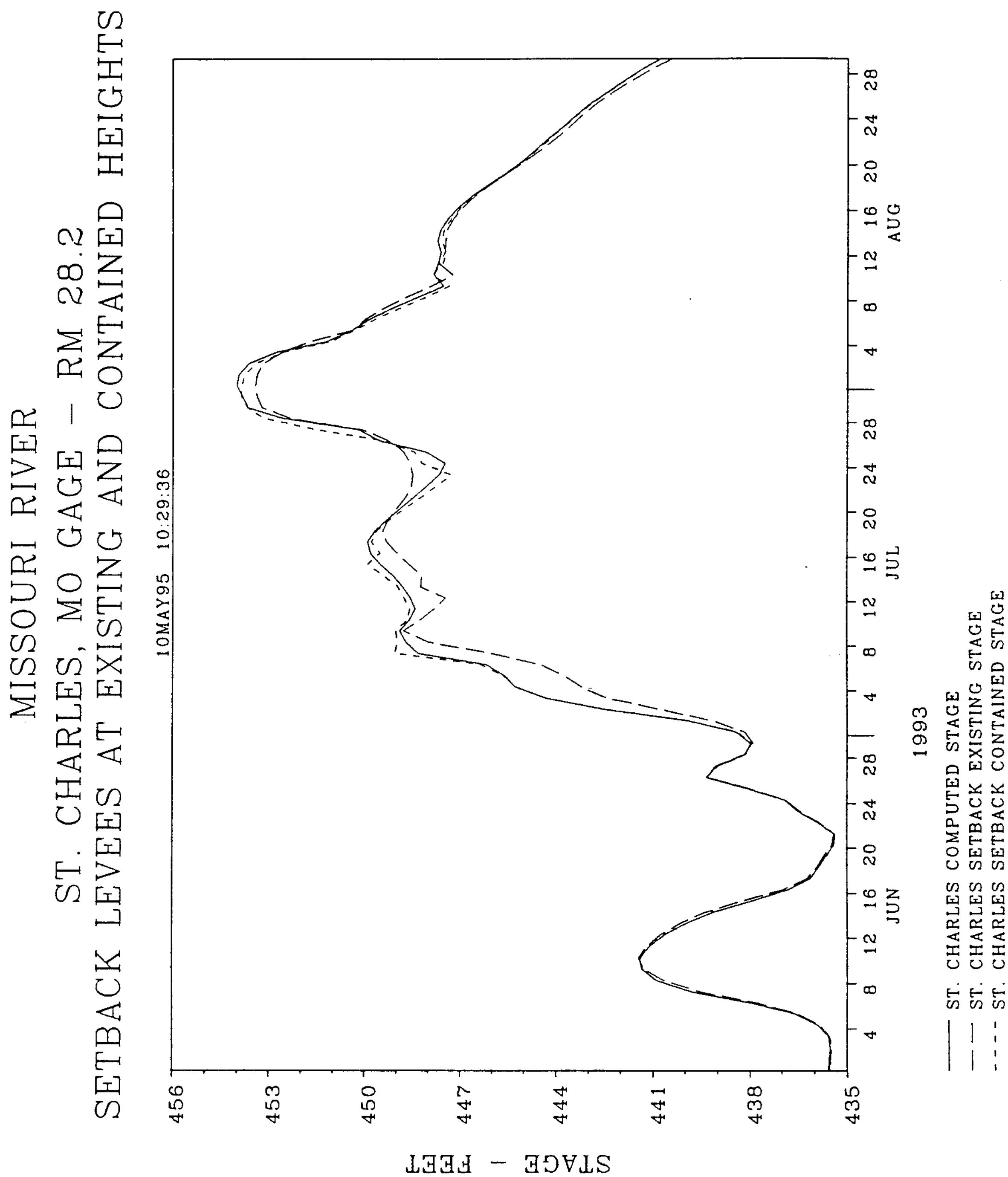

MISSOURI RIVER
ST. CHARLES, MO GAGE - RM 28.2
SETBACK LEVEES AT EXISTING AND CONTAINED HEIGHTS
10MAY95 10:29:36
STAGE - FEET
456
453
450
447
444
441
438
435
4
8
12
16
20
24
28
4
8
12
16
20
24
28
4
8
12
16
20
24
28
JUN
JUL
AUG
1993
ST. CHARLES COMPUTED STAGE
ST. CHARLES SETBACK EXISTING STAGE
ST. CHARLES SETBACK CONTAINED STAGE

PLATE SL-14.5